Transportation
Library
TA
1
.A526

TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

TRANSPORTATION LIBRARY
UNIVERSITY OF MICHIGAN

VOL. LII

JUNE, 1904.

Edited by the Secretary, under the direction of the Committee on Publications.

Reprints from this publication, which is copyrighted, may be made on condition that the full title of Paper, name of Author and page reference are given.

NEW YORK:

PUBLISHED BY THE SOCIETY.

1904.

TA
1
.A52 6

Entered according to Act of Congress, by the AMERICAN SOCIETY OF CIVIL ENGINEERS, in the Office of the Librarian of Congress, at Washington.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CONTENTS.

PAPERS.

No.		PAGE.
969	**SOUTH AFRICAN IRRIGATION.**	
	By **Francis Robert Johnson**	1
	Discussion on Paper No. 969:	
	By AUGUSTINE W. WRIGHT	27
	L. J. LE CONTE	30
	FRANCIS ROBERT JOHNSON	32
970	**DOCK IMPROVEMENTS AT LIVERPOOL.**	
	By **George Cecil Kenyon**	36
	Discussion on Paper No. 970:	
	By ALFRED NOBLE	65
	L. J. LE CONTE	66
	EMILE LOW	67
	GEORGE CECIL KENYON	69
971	**THE BREAKWATER AT BUFFALO, NEW YORK.**	
	By **Emile Low**	73
	Discussion on Paper No. 971:	
	By THOMAS D. PITTS	198
	WILLIAM T. LYLE	199
	D. E. HUGHES	202
	G. H. RAYMOND	203
	GEORGE E. FELL	205
	EMILE LOW	208
972	**A DESIRABLE METHOD OF DREDGING CHANNELS THROUGH RIVER BARS.**	
	By **S. P. Maximoff**	215
	Discussion on Paper No. 972:	
	By RUDOLPH HERING	238
	WILLIAM W. HARTS	239
	E. L. CORTHELL	240
	A. W. ROBINSON	247
	L. J. LE CONTE	249
	F. B. MALTBY	250
	WILLIAM GERIG	252
	F. H. HILLIARD	254
	S. P. MAXIMOFF	258
973	**THEORY OF THE SPHERICAL DOME WITH A HOMOGENEOUS SURFACE, AND OF THE FRAMED DOME; ALSO NOTES ON THE CONSTRUCTION OF MASONRY AND METAL DOMES.**	
	By **E. Schmitt**	262
	Discussion on Paper No. 973:	
	By IRVING P. CHURCH	309
	H. H. WADSWORTH	317
	E. SCHMITT	320

No. PAGE.

974 **METHOD USED BY THE RAILROAD COMMISSION, OF TEXAS, UNDER THE STOCK AND BOND LAW, IN VALUING RAILROAD PROPERTIES.**

By **R. A. Thompson** 328

Discussion on Paper No. 974:

By E. L. Corthell 346
F. Lavis 348
W. H. Coverdale 349
W. D. Taylor 353
R. A. Thompson 360

975 **FREEZING AS AN AID TO EXCAVATION IN UNSTABLE MATERIAL.**

By **James H. Brace** 365

Discussion on Paper No. 975:

By George E. Thomas 437
D. E. Moran 438
E. L. Abbott 442
T. Kennard Thomson 442
Werner Boecklin 443
H. J. Campbell 444
A. Gobert 446
Charles Sooysmith 447
James H. Brace 449

976 **THE SUBSTRUCTURE OF MARSH RIVER BRIDGE.**

By **Herbert J. Wild** 451

Discussion on Paper No. 976:

By George B. Francis 465

977 **THE LOCATION OF THE KNOXVILLE, LA FOLLETTE AND JELLICO RAILROAD, OF THE LOUISVILLE AND NASHVILLE SYSTEM.**

By **W. D. Taylor** 467

Discussion on Paper No. 977:

By Emile Low 490
William P. Watson 497
E. J. Beard 505
Walter Watson 510
William G. Raymond 517
F. Lavis 519
W. H. Coverdale 521
Harvey Linton 526
W. T. Forsythe 527
W. D. Taylor 529

MEMOIRS OF DECEASED MEMBERS.

James Peter Bogart, M. Am. Soc. C. E. 545
Charles Edward Henry Campbell, M. Am. Soc. C. E. 547
Benjamin Burgh Smith, M. Am. Soc. C. E. 550
Selwyn Mellon Taylor, M. Am. Soc. C. E. 552
Paul Davis Cunningham, Assoc. M. Am. Soc. C. E. 556
William Ward Reed, F. Am. Soc. C. E. 558

PLATES.

PLATE.		PAPER.	PAGE.
I.	Plan of Liverpool Dock Estate	970	37
II.	Plan of Mersey Dock Estate	970	41
III.	General Plans of Entrances, Locks, etc., Brunswick Dock	970	43
IV.	New River Entrance to Brunswick Dock	970	45
V.	Map of Buffalo Harbor, and Vicinity	971	75
VI.	Old Breakwater. Building Concrete Superstructure	971	107
VII.	Old Breakwater. Building Timber Superstructure and Setting Concrete Blocks	971	109
VIII.	The Stone Breakwater at Buffalo, N. Y.	971	123
IX.	The Stone Breakwater after Settlement	971	137
X.	Building Concrete Superstructure of the South Harbor Section	971	167
XI.	Concrete Superstructure of the South Harbor Section	971	175
XII.	Junction of Stone and Timber Sections; and South Pier	971	179
XIII.	Building Superstructure of the North Breakwater, Buffalo, N. Y.	971	185
XIV.	Typical Russian Passenger Steamboat and Tugboat	972	219
XV.	Typical Russian Wooden Barges and "Bielana"	972	221
XVI.	Map of European Russia	972	225
XVII.	A Typical Russian Ladder Dredge	972	229
XVIII.	The Ladder Dredge *Volgian 9th*	972	231
XIX.	Marsh River Bridge: East Abutment and Old Bridge	976	459
XX.	Marsh River Bridge: Driving Long Piles, Timber Grapples, etc.	976	461
XXI.	Plan and Profile of Part of the Knoxville, La Follette and Jellico Railroad	977	477
XXII.	Rockhead in Big Creek Gap, Cumberland Mountain	977	535
XXIII.	Views, Construction of Knoxville, La Follette and Jellico Railroad.	977	537
XXIV.	Views, Construction of Knoxville, La Follette and Jellico Railroad.	977	539

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 969.

SOUTH AFRICAN IRRIGATION.*

By Francis Robert Johnson, M. Am. Soc. C. E.

With Discussion by Messrs. Augustine W. Wright, L. J. Le Conte and Francis Robert Johnson.

The Present Position of Irrigation.

In South Africa it may be said that the limit of private enterprise in the construction of irrigation works has only been reached as far as leading the ordinary flow of the smaller streams is concerned. The ditches, or furrows, as they are called, in many cases have been made by the farmers without any levels, the usual procedure being to test the furrow, as construction proceeds, by letting the water follow up the work. The result is that the fall is generally insufficient, and a great deal of silt is deposited at flood time.

As a rule, a loose rock weir, from 1 to 5 ft. high, as required, is thrown across the stream at the point of intake. The following are typical colonial furrows, the figures having been determined by the writer from actual survey.

Case 1.—Length, 5 miles. Flow in furrow near fields, in average condition, 1.98 cu. ft. per second. Waters 616 acres. Flow of river, observed during one summer: minimum, 11 cu. ft. per second; maximum, 1 280 cu. ft. per second. Fall of river, about 20 ft. per mile. Greatest known flood, about 5 800 cu. ft. per second. Catchment area, 239 sq. miles. Average annual rainfall at source, 50.71 ins.;

* Presented at the meeting of October 7th, 1903.

and, in vicinity of fields, 21.06 ins. Seasonal rainfall: Spring, 4.73 ins.; summer, 8.82 ins.; autumn, 5.56 ins.; winter, 1.95 ins. Wheat is largely grown from the middle of March to the middle of December, during which period the rainfall averages 11.75 ins., which is supplemented by irrigation. Other crops are maize, Kaffir corn and potatoes.

Case 2.—A group of five furrows. Maximum length, about 7 miles. Average combined flow near fields, during three summer months, 4.33 cu. ft. per second, or 17 000 000 cu. ft. for the whole summer, exclusive of the night flow which is seldom used. Waters 718 acres, all the year, and an additional 350 acres in the winter during good seasons. Flow of river, observed during one summer: minimum, 0.07 cu. ft. per second; maximum, 1 378 cu. ft. per second. Fall of river, about 45 ft. per mile. Greatest known flood, about 5 200 cu. ft. per second. Catchment area, 68 sq. miles. Average annual rainfall at source, 23.25 ins., and, in vicinity of fields, 7.21 ins. Seasonal rainfall: spring, 2.02 ins.; summer, 1.43 ins.; autumn, 2.76 ins.; winter, 1.00 in. Lucerne is largely grown as food for ostriches; other crops are tobacco, wheat, barley, maize, beans, vines and fruits.

There is considerable irrigation from springs, or "fountains," as they are called, but, unfortunately, there are no statistics of any kind available, and even in the case of furrows there is only a very partial record which refers mostly to those used for village water supplies. The finding and opening up of two of these "fountains" will now be described, as they are rather interesting. In both cases the sites were inspected and the details noted and checked on the ground, as far as possible.

Case 3.—A farmer purchased a farm of 4 000 acres on which there was no water. After a time he noticed signs of moisture in a hollow, which he therefore plowed over regularly so as to assist the summer thunderstorms in forming a sluit (stream) and opening up the ground. This resulted in the rock being laid bare, eventually, at a depth of 20 ft. from the surface, and, as the flow of water was then stronger, and a trap dike ran across the country just below, he decided to have bores put down.

Four 2½-in. bore holes were drilled 40 ft. into the rock, and iron pipes were laid from them to a small reservoir near to, and commanding, the cultivated ground, the pipes being 2 ins. in diameter and having a considerable slope. The flow, in all, is said to be equal

to 160 000 galls. in 24 hours, say 25 600 cu. ft. In 10 days this would enable 50 acres to get a watering of 1¼ ins.

The farmer uses this supply over an area of 140 acres, and says it is enough to provide against drought, and save the crops in bad seasons, while it increases the yield very much in good seasons.

Case 4.—A farmer, in this case, also, noticed moisture above a trap dike, and excavated what in appearance is a miniature railroad cutting, with slopes of about ½ to 1, formation width 2 to 3 ft., and depth about 10 ft. to the rock. Three holes, about 1½ ins. in diameter, were bored in the rock to a depth of only 6 ft., and from each hole water boils up to a height of several inches, and runs down the cutting to a small reservoir, where it is stored.

There are a great many of these fountains, both in the Cape and Orange River Colonies, and it is believed there are also a good many in the Transvaal.

Many farms have one or more small reservoirs to catch surface water. To avoid risk, these are usually located as near the watershed as possible, consistent with getting a catchment at all. These are sometimes used for watering cattle, and sometimes to irrigate gardens. Very few hold as much as 500 000 cu. ft.

Occasionally, dams are thrown across a well-defined stream, but owing to defective design and construction, or want of an adequate waste weir, many of these have burst, and their ruins can be seen in many parts of the country.

Successful storage works, of any magnitude, for irrigation, are very scarce. There are also very few wells used for irrigation in the Cape or Orange River Colonies; but in Natal the free Indians, mostly from Madras and Bengal, have excavated some, and lift the water with picotahs to irrigate their small patches of ground, on which these industrious people raise very fine vegetables. To sum up, it may be said that leadings have only been taken from the smaller streams which run near the surface of the country, and hardly at all from the larger ones and those which run at a considerable depth below the surface, while wells and large storage works for irrigation are almost unknown.

Geology.

The geology of South Africa is somewhat puzzling, as it has not yet been worked out thoroughly, and, indeed, cannot be until a better

map is available. In addition, many contradictory statements, based on hurried visits to the country, have been made by geologists, and these have only served to complicate matters.

As far as known, intrusive granite rocks appear to form the basement of the country. They are exposed at Cape Town at the sea level, in contact with the overlying series of slates and compact quartzites, called, in the Cape, "Malmsbury beds," which are the oldest, known sedimentary rocks thus far discovered.

Again, in the Transvaal, at an elevation of some 5 800 ft., granite is exposed here and there over considerable areas north of Johannesburg. It is also exposed at the base of the Drekensburg Range in the De Kaap Valley, and in the bed of the Letaba River, north of the Murchison Hills.

It would appear that there had been considerable denudation throughout the country, followed by upheaval and then by more denudation; also, that the denudation increased from north to south, and that the débris traveled southward. As a result of the general upheaval, the slates and quartzites, not only in the Cape, but in the Orange River Colony and the Transvaal, have generally a very high angle of dip.

Peculiar throughout South Africa are the intrusive trappean dikes, which penetrate, not only the granites and the slates, but other formations, although, thus far, they have not been noticed in the Table Mountain sandstones, a series of rocks at the Cape which lies unconformably on the upturned edges of the Malmsbury beds. Locally, where these trappean dikes have been intruded, and they are very numerous indeed, there has been, naturally, much disturbance and alteration in the overlying rocks. The dikes are generally diabase and dolerite in the Cape, but are said to be mostly diorites in the Transvaal.

The slates, generally speaking, are rotted to a considerable depth. The writer has found this to be the case 30 ft. from the surface, and streams running over these rocks often disappear, only to reappear at some intrusive trap dike which bars the way to further underground movement of the water. The late W. H. Furlonge, M. Am. Inst. Min. Engrs., mentions* that the granite of the De Kaap Gold Fields is decomposed to a depth of 200 ft.

* *Transactions*. Am. Inst. Min. Engrs., Vol. XVIII, 1890, p. 337.

In the usually flat-bedded series of rocks, such as the sandstone series, and the coal formation, alternate hard and soft layers are very common, and in many of the streams which flow over these rocks the writer has noticed that the bed has been more or less blown up, owing to the water getting into a soft layer below the surface.

It is difficult at present to say much as to the proper sequence of the rocks which lie above the Malmsbury beds at the Cape, and above the slates in the Transvaal, because the geology of all the colonies has not yet been treated as a whole, but it may be said that these rocks are often much altered and contorted as a result of the dynamic movements which have taken place, and that the origin of some of them has not yet been accounted for satisfactorily. It may be mentioned, however, that while, in the Cape, the Table Mountain sandstones lie unconformably on the upturned edge of the Malmsbury beds, in the Transvaal it is stated that the sandstone series there lies conformably on the slates.

The soil covering, on the whole, is thin and poor in quality. There are exceptions, of course, but, comparatively speaking, these are of small area, and are scattered widely. It is probable that, as in many other countries, for instance, the United States of America, the soils of the arid districts, particularly in the Cape and Orange River Colonies, will be found to be the most fertile; indeed, in the Cape Karoo the writer has noticed this fertility as being most marked in the few places where water has been applied. To a great extent this is, doubtless, due to the presence of lime and various salts in considerable quantities which the scanty rainfall has been unable to wash out. The lime is in the form of a very fine powder, and is conspicuous to the eye.

The following difficulties are likely to present themselves to the hydraulic engineer, in many cases:

1.—Indifferent foundations for large masonry dams.

2.—Scarcity of good puddle.

3.—Leaky reservoirs and canals.

In the writer's opinion, the method of construction for storage works most suited to the country, on the whole, will be found to be concrete core-walls carried well down, with rock-filled outer slopes, and inner slopes of earth, not only from the geological point of view, but from the economical one also.

The Seasons.

The seasons are opposite to those of Europe, *viz.:*

Spring: September, October, November;

Summer: December, January, February;

Autumn: March, April, May;

Winter: June, July, August.

The mean temperatures at typical places in the Colonies, during the various seasons, in the year 1899, are shown in Table No. 1.

TABLE No. 1.—Mean Temperature at Typical Places in the Colonies During the Various Seasons of the Year 1899.

Place.	Colony.	Elevation.	Spring.	Summer.	Autumn.	Winter.
			°	°	°	°
Port Elizabeth.....	Cape Colony..	Sea Level.	62.1	69.6	65.3	59.9
Somerset East.....	Cape Colony..	2 400 ft.	62.7	71.6	63.4	56.1
Steynsburg.....	Cape Colony..	4 850 ft.	60.9	70.3	57.2	48.4
Durban.....	Natal.....	Sea Level.	71.6	77.8	70.3	67.4
Maritzburg.....	Natal.....	2 218 ft.	66.8	72.7	64.9	59.4
Ladysmith.....	Natal.....	3 284 ft.	69.8	75.8	65.8	53.1
Bloemfontein.....	Orange River.	4 518 ft.	65.2	74.4	59.5	50.6
Johannesburg.....	Transvaal.....	5 735 ft.	....	....	58.4	51.8

In Maritzburg, Natal, the winter temperature would ripen cereals if the days were long enough, as it exceeds the summer temperature of Calgary, Alberta, in the Dominion of Canada, the latter being only 58.8 degrees. At Calgary, wheat and oats ripen in this temperature, which shows the potent effect of light, as there are 17 hours of daylight at Calgary, as compared with some 11 to 12 hours during the winter at Maritzburg.

The Rainfall.

Tables Nos. 2 and 3 give the seasonal and annual rainfall for places located along the drainage lines of the country.

These records have been compiled by the writer from various sources,* and speak for themselves in showing how much irrigation is needed; but, if all the variations from the mean could have been shown, they would have afforded a still more potent argument.

* (1) Buchan's "Discussion of South African Rainfall" (compiled from Cape Government Statistics.
(2) Mining Records (Transvaal).
(3) Natal Government Statistics.

TABLE No. 2.—Rainfall in Cape Colony.

Hydrographic district No.	Main river.	Source.	Average Rainfall, in Inches.					Number of years in period.	Midway.	Average Rainfall, in Inches.					Number of years in period.
			Seasonal.				Annual.			Seasonal.				Annual.	
			Spring.	Summer.	Autumn.	Winter.				Spring.	Summer.	Autumn.	Winter.		
1	Buffels	Garies	1.31	0.20	1.80	2.43	5.74	7	Springbokfontein	1.99	0.84	3.32	3.59	9.74	10
2	Breede	Ceres	9.35	2.45	11.85	18.00	41.65	10	Robertson	3.56	2.08	3.38	4.01	13.03	10
2	Great Berg	Wellington	5.17	1.88	7.31	11.22	25.58	10	Piquetberg	6.05	1.40	6.02	8.89	22.36	10
2	Olifants	Matjesfontein	1.52	1.27	1.97	1.67	6.43	10	Clanwilliam	2.16	0.78	2.75	3.72	9.41	10
3	Hartebeest	Sutherland	2.38	1.82	3.46	2.35	10.01	10	Rheboksfontein	1.45	2.21	2.43	1.02	7.11	6
4	Gouritz	Beaufort West	2.53	3.51	2.96	0.74	9.74	10	Calitzdorp	2.68	1.58	2.91	1.23	8.40	10
5	Sundays	Graaf Reinet	5.29	5.86	5.85	1.76	18.76	10	Jansenville	3.39	4.46	4.09	1.13	13.07	10
5	Gamtoos	Brakfontein	2.36	4.30	4.16	0.94	11.76	10	Willowmore	2.47	3.26	3.89	1.42	11.04	10
6	Brak	Victoria West	2.48	5.43	3.35	0.80	12.06	10	Britstown	2.61	3.49	4.86	1.09	12.05	7
7	Kraal	Molteno	5.87	7.02	9.53	5.28	27.70	6	Barkly East	6.46	11.20	7.65	3.05	28.31	10
7	Sea Cow	No record	.....	.....	.....	.....	.....	.....	Boschfontein	3.61	4.65	5.02	1.17	14.45	3
8	Great Salt	Steynsburg	4.20	7.88	6.10	2.19	20.28	10	Witmoss	4.57	6.29	5.29	2.06	18.21	10
9	Great Kei	Sterkstrom	5.36	9.74	6.23	2.46	23.79	10	Keilands	7.90	10.55	6.96	2.27	27.66	5
10	Umzimvubu	Mount Fletcher	11.06	14.86	7.73	1.56	35.21	3	Gumba	10.27	12.96	8.63	2.25	34.11	6
11	Malopo	No record	.....	.....	.....	.....	.....	.....	No record	.....	.....	.....	.....	.....	.....
12	Groen Water	No record	.....	.....	.....	.....	.....	.....	Griquatown	2.61	6.92	5.62	1.44	16.59	10
12	Harts	No record	.....	.....	.....	.....	.....	.....	Vryburg	2.48	16.04	5.75	1.23	25.50	4

TABLE No. 3.—Rainfall in Orange River, Transvaal and Natal Colonies.

Hydrographic district.		Average Rainfall, in Inches.					Length of record, in years.		Average Rainfall, in Inches.					Length of record, in years.
		Seasonal.				Annual.			Seasonal.				Annual.	
		Spring.	Summer.	Autumn.	Winter.				Spring.	Summer.	Autumn.	Winter.		
ORANGE RIVER COLONY.														
Caledon River	Midway—Ladybrand	8.44	14.32	8.62	3.02	34.41	8	Outfall—Bethulie	4.25	9.00	6.19	2.36	21.80	8
Riet River	Midway—No record	.....	.....	.....	.....	.....	.....	Outfall—Jacobsdal	3.82	7.97	6.25	1.72	19.76	10
Modder River	Midway—Bloemfontein	5.58	10.08	7.89	1.84	25.39	10	Outfall—Jacobsdal	3.82	7.97	6.25	1.72	19.76	10
Zand River	Midway—No record	.....	.....	.....	.....	.....	.....	Outfall—Hoopstad	4.35	12.22	5.87	0.91	23.35	4
Valsch River	Midway—Kroonstad	6.73	11.85	7.57	1.02	27.17	10	Outfall—No record	.....	.....	.....	.....	.....	.....
Rhenoster River	Midway—No record	.....	.....	.....	.....	.....	.....	Outfall—No record	.....	.....	.....	.....	.....	.....
Wilge River	Source—Harrismith	8.94	12.28	6.38	1.54	29.14	5	Midway—No record	.....	.....	.....	.....	.....	.....
Klip River	Source—Newcastle	5.39	17.89	6.69	0.42	30.39	3	Midway—No record	.....	.....	.....	.....	.....	.....
TRANSVAAL COLONY.														
.....	Johannesburg	6.25	16.25	6.50	0.75	29.75	11	.....	.....	.....	.....	.....	.....	.....
NATAL COLONY.														
Umzimkulu River	Midway—Umzimkulu	8.68	11.98	6.02	1.72	28.40	9	Outfall—Shepstone	14.52	16.66	8.69	3.27	43.14	4
Umkomas River	Midway—Richmond	9.39	19.22	8.70	1.98	39.29	5	Outfall—No record	.....	.....	.....	.....	.....	.....
Umgeni River	Source—Howick	7.23	14.28	8.16	0.96	30.63	4	Midway—Maritzburg	7.90	17.90	8.66	1.21	35.68	6
Umvoti River	Source—Greytown	9.93	15.51	6.19	1.20	32.83	5	Outfall—Stanger	11.58	13.12	9.21	3.58	37.49	5
Tugela River	Source—No record	.....	.....	.....	.....	.....	.....	Midway—Ladysmith	5.57	14.15	13.30	0.75	33.77	3
Buffalo River	Source—Newcastle	5.39	17.89	6.69	0.42	30.39	3	Midway—No record	.....	.....	.....	.....	.....	.....

The writer, however, will give one instance which has quite convinced him that farming without irrigation, under the erratic rainfall conditions of South Africa, is no pastime. The average summer rainfall for Maritzburg is 17.90 ins., but during 1901–02 it was 10.62 ins., and during the season of 1902–03 it has been only 6.29 ins., with the result that no crop could be grown without artificial watering, and everything sown under rainfall conditions was a failure. It is unfortunate that no hydraulic engineer has yet had the opportunity to discuss the detailed rainfall records of South Africa from his own point of view, for there is much to be learned which the ordinary reports do not deal with, such as seasonal rainfall, intensity of precipitation, droughts, variableness of the rainfall, etc., etc.

The writer, however, hopes that, as regards seasonal rainfall, which is so important in considering irrigation, he has helped to fill the gap, as far as the records at his disposal allowed.

TABLE No. 4.—RAINFALL AT MARITZBURG, NATAL, FOR 6 YEARS, 1895–1900; AND AT THE WRITER'S FARM, 2 MILES FROM THE CITY, DURING PART OF THE PERIOD, 1902–03.

Season.	1895	1896	1897	1898	1899	1900	Average.	1902	1903
Spring.....	5.64	8.54	7.00	10.28	9.86	6.11	7.91	3.64	.....
Summer.....	30.98	17.06	13.28	19.37	15.31	11.41	17.90	10.62	6.29
Autumn.....	8.73	7.80	9.95	13.43	5.63	6.42	8.66	7.21	.....
Winter.....	0.62	2.42	1.43	0.56	0.98	1.25	1.21	2.89	.....

The writer would advocate strongly the use of seasonal, instead of monthly, or annual, rainfall records in dealing with irrigation problems, as they show in a much clearer manner what, if any, artificial watering is required for any particular crop.

In the writer's report on colonial irrigation,* forms were provided which included the seasonal rainfall among other details to be recorded.

THE WATER SUPPLY.

The writer has compiled in Tables Nos. 5, 6 and 7 lists of the principal rivers of South Africa, together with facts which are likely to be useful to the hydraulic engineer.

* Blue Book, G 76, of 1899.

TABLE No. 5.—Rivers of Cape Colony.

Hydrographic District.	Name of river.	Source.	Outfall.	Length in miles.	Rainfall, in Inches.	
					Source.	Midway.
1..	Buffels.....	Langeberg Mts.....	Atlantic Ocean..	170	5.74	9.74
2..	Breede.....	Ceres.....	Indian Ocean....	140	41.65	13.03
2..	Great Berg..	Paarl.....	Atlantic Ocean..	100	25.58	22.36
2..	Olifants.....	Bonteberg Mts.....	Atlantic Ocean..	215	6.43	9.41
3..	Hartebeeste.	Nieuwveld Mts.....	Orange River...	370	10.01	7.11
4..	Gouritz.....	Nieuwveld Mts.....	Indian Ocean....	200	9.74	8.40
5..	Sundays.....	Sneeuwberg Mts.....	Indian Ocean....	200	18.76	13.07
5..	Gantoos.....	South of Victoria West	Indian Ocean....	260	11.76	11.04
6..	Brak.....	Near Victoria West...	Orange River...	160	12.06	12.05
7..	Kraal.....	Stormberg Mts.....	Orange River...	115	27.70	28.31
7..	Sea Cow.....	Sneeuwberg Mts.....	Orange River...	110	No record.	14.45
8..	Great Salt...	Zuurrberg Mts.....	Indian Ocean....	265	20.28	18.21
9..	Great Kei...	Stormberg Mts.....	Indian Ocean....	160	23.79	27.66
10..	Umzimvubu	Drakensberg Mts.....	Indian Ocean....	150	35.21	34.11
11..	Malopo.....	Kalahari Desert.....	Orange River...	380	No	record.
12..	Groen Water	Mount Huxley.....	Orange River...	115	No record.	16.59
12..	Harts.....	Transvaal.....	Vaal River.....	140	No record.	25.50

In addition, the Orange and Vaal Rivers form boundaries between the Cape, Orange River and Transvaal Colonies. The former is approximately 1 100, and the latter 600 miles, in length. Mention should be made here of the great mountain of the Drakensberg Range, "Mont aux Sources," 11 000 ft. in height, from the base of which four important rivers take their source, viz., the Wilge River flowing north, the Tugela River flowing east, the Orange River flowing south, and the Caledon River flowing west.

The Vaal River also rises in the Great Drakensberg Range, but further north.

TABLE No. 6.—Rivers of Orange River Colony.

Name of River.	Source.	Outfall.	Length, in miles.	Rainfall, in Inches.		
				Source.	Midway.	Outfall.
Caledon.....	Mont aux Sources..	Orange River...	235	.....	34.41	21.80
Riet.....	Beyers Bergen.....	Vaal River.....	190	.....	.....	19.76
Modder.....	Beyers Bergen.....	Riet River.....	170	.....	25.39	19.76
Zand.....	Klokolani.....	Vaal River.....	180	.....	.....	23.35
Valsch.....	Witte Bergen.....	Vaal River.....	155	.....	27.17	.....
Rhenoster.....	Elands Kop.....	Vaal River.....	100	No record.		
Wilge.....	Mont aux Sources..	Vaal River.....	150	29.14	.....	.....
Klip.....	Drakensberg Mts...	Vaal River.....	80	30.39	.....	.....

Rivers of Transvaal Colony.—The Transvaal has not been as well mapped as the other colonies, particularly the northern portion, which contains the principal streams. Generally speaking, a line drawn east and west through Johannesburg would represent, approximately, the divide between the Vaal and Limpopo Rivers, and south of that line the streams, comparatively speaking, are insignificant.

TABLE No. 7.—RIVERS OF NATAL COLONY.

Name of river.	Source.	Outfall.	Length, in miles.	Rainfall, in Inches.		
				Source.	Midway.	Outfall.
Umzimkulu	Drakensberg Mts.	Indian Ocean	110	.....	28.40	43.14
Umkoman	Drakensberg Mts.	Indian Ocean	100	.....	39.29	.....
Umgeni	Drakensberg Mts.	Indian Ocean	85	30.63	35.08	42.46
Umvoti	Drakensberg Mts.	Indian Ocean	65	32.83	.....	37.49
Tugela	Mont aux Sources	Indian Ocean	210	.....	33.77	.....
Buffalo	Drakensberg Mts.	Tugela River	130	30.39	.....	.....

Snow falls more or less on the principal mountain ranges every winter, particularly the Drakensberg, Stormsberg, Sneeuwberg and Langeberg.

The writer has estimated* that approximately 5% of the rainfall, as flowing waters, plus, possibly, 1% as ground-waters, might be available in the Cape Colony for irrigation; the mean annual rainfall varying from 6 to 33 ins. for the various hydrographic districts.

Allowing 2 acre-feet in the fields, as the duty of water, the supply worked out as sufficient for 4% of the total area of the Colony for which statistics were available (*viz.*, Districts 1 to 10), say 5 000 000 acres.

In the light of present knowledge it is difficult to say whether this was under- or over- estimating the case.

The writer, however, has since seen an estimate† made for the arid districts of the United States in which the supply there is stated as sufficient for 74 000 000 acres, the total land surface being 892 000 000 acres; or, practically, for 8% of the area of the arid districts in the United States, as against 4% for ten out of the twelve hydrographic districts of Cape Colony.

* "Special Report on Colonial Irrigation, and Hydrographic Survey, with Statistics." Cape of Good Hope Blue Book, G 7f, 1899.

† "Irrigation in the United States," by Frederick Haynes Newell, Chief Hydrographer, U. S. G. S., 1902.

Owing to the scarcity of rainfall records, it was not found possible to estimate for Districts 11 and 12, and therefore they were excluded.

The writer understands from Mr. Newell's book, however, that in the United States at the present time the total area actually irrigated does not equal 1% of the land surface. Very few flow measurements have been made in South Africa, therefore the following records may be found to be of interest:

Hydrographic District No. 4, Cape Colony.

Principal river basin	Gouritz River.
Branch stream	Nels River.
Catchment area, in square miles	68
Extreme length, in miles	11
Mean width, in miles	6.19
Summer rainfall, in inches (one season, '97–98)	3.07
Period of measurement of flow, Nov. 21st, '97, to Feb. 21st, '98.	
Discharge, in millions of cubic feet	55.8
" in acre-feet	1 280
Estimated maximum flood, in cubic feet per second.	5 200
Observed " " " " "	1 378
Observed maximum flood, in cubic feet per second per square mile	20.25
Observed minimum flow, in cubic feet per second.	0.077
Coefficient of discharge, for summer	0.115

The coefficient of discharge is referred to the rainfall at the lower end of the catchment. If referred to the gauge on the mountains, it would be only 0.056.

The river was cross-sectioned very carefully, and velocities were measured by floats at various stages of the stream. The heights were measured at a gauge post at irregular intervals, but the results are believed to be fairly accurate.

Hydrographic District No. 8, Cape Colony.

Principal river basin	Great Salt River.
Branch stream	Theebus River.
Catchment area, in square miles	78.84
Extreme length, in miles	11.62
Mean width, in miles	6.79

Rainfall, in inches, for one year, Oct., '96, to Sept., '97	13.75
Period of measurement of flow, Oct., '96, to Sept., '97.	1 year.
Discharge, in millions of cubic feet	141½
" in acre-feet	3 240
Observed maximum flood, in cubic feet per second.	379
" " " in cubic feet per second per square mile	4.8
Observed minimum flow, in cubic feet per second.	0.23
Coefficient of discharge, for the year	0.055
" " " " spring	0.016
" " " " summer	0.065
" " " " autumn	0.050
" " " " winter	0.076

The coefficients of discharge are referred to the mean of nine to fifteen rain gauges located over the catchment. The flow was measured very carefully over a 20-ft. rectangular notch, and, in addition, a 12-in., V-notch was used for small discharges.

Hydrographic District No. 8, Cape Colony.

Principal river basin	Great Salt River
Branch streams	Theebus and Rhenoster Rivers
Catchment area, in square miles	145.58
Extreme length, in miles	19.5
Mean width, in miles	7.47
Rainfall, in inches, for 1898	19.36
Period of measurement of flow, Jan. 1st to Dec. 31st, '98	1 year.
Discharge, in millions of cubic feet	702
" in acre-feet	16 119
Observed maximum flood, in cubic feet per second.	11 150
" " " in cubic feet per second per square mile	76.6
Observed minimum flow, in cubic feet per second.	0.42
Coefficient of discharge, for the year	0.107
" " " " spring	0.065
" " " " summer	0.147
" " " " autumn	0.028
" " " " winter	0.257

The coefficients of discharge are referred to the mean of twenty-two to twenty-three rain gauges located over the catchment. The flow was measured carefully over one 12-in. V-notch, two 20-ft. rectangular notches, and, for extreme floods, a channel 150 ft. in length. During January, 1898, the average rainfall over the catchment was 6.45 ins., while the previous average in the village for 9 years was only 2.62 ins. On December 6th, 1898, 3.07 ins. fell, the average for the month of December for 20 years being 1.96 ins.

The measurements for District No. 8 were not made by the writer. They are believed to be thoroughly reliable.

The Crops.

The principal ordinary crops grown in South Africa are as follows: Maize, Kaffir corn (*Sorghum vulgare*), oats, barley, potatoes, pumpkins, beans and lucerne.

These are the standard crops, grown more or less by almost every farmer, while wheat and turnips are only grown in certain districts. Excluding the comparatively small portion of the Cape Colony which is subject to good winter rains, the periods of sowing and the number of months taken to mature crops are generally as shown in Table No. 8.

TABLE No. 8.—Period of Sowing Crops, Etc.

Crop.	Period of sowing.	Time taken to mature.
Maize	October to January	3 to 4 months.
Kaffir corn	" "	5 to 6 "
Oats	April to June	6 "
Barley	" "	6 "
Potatoes	August to February	3 "
Pumpkins	November to January	3 "
Beans	December or January	3 "
Lucerne	Spring or autumn	(Perennial.)

As a rule, oats, barley and lucerne are grown under irrigation, and the others generally as dry crops, but the maize and Kaffir corn would benefit greatly by being irrigated also. That this is not done is due to the fact that they are grown on a much larger scale than the other crops, and usually on lands which are not capable of being watered from the small irrigation systems of most farmers. Oats and barley

can be grown during the summer months, but as they almost invariably rust at that time, the winter crop is preferred.

The early and late crops of potatoes are the most valuable; the former, to catch the market at a time of the year when they are very scarce throughout South Africa, and the latter, to keep as long as possible through the winter. Maize and Kaffir corn are generally grown for their grain, and the stalks are given to the cattle afterward. Oats and barley are grown for forage; the former is generally sold in pressed bales as oat hay. The latter is usually sold green after it has come into head. Lucerne is sometimes used green for fodder, and sometimes made into hay and baled.

As a rule, the yield in South Africa is poor. The following are considered fairly good crops under favorable rainfall conditions:

Maizé (corn) ½ ton per acre, say 16 bushels;
Oats (forage) 1 ton per acre;
Potatoes, 3 tons per acre, say 127 bushels.

The usual prices obtained for good samples are as follows, depending on the supply:

Maize (corn)	per 100 lbs.	6s.	to 11s.
Oats (forage)	"	6s. 6d.	to 10s. 6d.
Potatoes	"	7s.	to 14s.
Lucerne (hay)	"	8s. 6d.	to 10s. 6d.

Maize, Kaffir corn, pumpkins, and potatoes will not stand any frost, while oats, barley, and lucerne are not injured thereby. Frosts may occur anytime between April and August, and occasionally in September and October. They are sometimes very severe, depending on the altitude, and often ruin the fruit crops in high places, like the Stormberg District in the Cape Colony.

Apple and pear trees bloom in August and September, and the fruit is ripe in December to March. Citrus fruits, such as oranges and lemons, are ripe in June to September, grapes in January to March, and peaches and Japanese plums in January and February. Grapes are grown on a very large scale in vineyards, for making wine and brandy, in the district subject to winter rains (Western Province of Cape Colony), and are usually irrigated.

Elsewhere, with summer rains, which spoil the quality, they are only grown on a small scale in private gardens.

With regard to the water supply required to produce a crop, the following may be looked on as a minimum; but, for full development, double the quantity is not too much in the case of cereals and lucerne:

Maize and Kaffir corn	18 ins.
Oats and barley (winter crop)	9 "
Potatoes	15 "
Lucerne	24 "
Apple trees	15 "
Grape vines	18 "

The writer would strongly advocate all crops being sown in drills with wide intervals, instead of either broadcast or in drills with narrow intervals, as generally practised. He has experimented a good deal in this direction, and believes the following distances will usually result in a yield quite up to the average, while the advantages, in the way of enabling water to be easily applied, are very great.

Maize for Corn.—Six feet between the drills and plants 3 ft. apart in the drill.

Maize for Fodder.—Three feet between the drills, and continuously in the drill.

Oats and Barley.—Three feet apart between the drills, and continuously in the drill, with the usual allowance of seed per acre.

Lucerne.—Two feet apart between the drills, and continuously in the drill.

In the latter case the writer commenced with 1½-ft. intervals, but, finding that the outside rows always came up about 50% higher than the others, he increased the distance to 2 ft.

The South African Farmer.

The South African farmer carries on his art under difficulties which would appal many an English farmer, notwithstanding the latter's troubles in the way of low prices, bad weather, and occasional shortage of labor.

The weather and labor troubles are serious enough in South Africa, probably worse than in England, but, in addition, there are many other things, such as insects and fungus pests, very dangerous cattle diseases, and, in the majority of cases, want of transport facilities, and farms far too large for anyone with an ordinary amount

of capital to work properly. Implements, good seed, and manure are also high in price, and at times unobtainable.

When the war broke out, in the beginning of October, 1899, that is 3½ years ago, it practically put a stop to all professional work for the Civil Engineer, and that state of affairs has continued to the present day. Under these circumstances the writer has learned his lesson in South African farming in that greatest of all schools "Practical Experience." That experience was undertaken with two objects in view, the first being to try and reduce the cost of living in the most expensive country in the world during a very trying time, and the second to gain some knowledge of the needs of the farmer as regards irrigation. The writer's opportunities, like his water supply, have been limited, but he has learned much, and he hopes a few of his gleanings from his farm of 40 acres in Natal, may be of interest to others.

The majority of insect pests are worse in seasons of drought, when the plants are least able to withstand their ravages, and it is probable that plentiful irrigation would, in most cases, locusts excepted, result in a crop in spite of them.

The following are the writer's notes as to the water supply necessary to give a crop, but, of course, the question is complicated by that unknown factor, the quantity of moisture in the ground, at the time of sowing, of which the plants can avail themselves.

Maize.—Sown on sloping ground in rows 3 ft. apart, on November 4th, 1902. Each row 2 Gunter's chains in length. Rather dry, red soil. Rainfall, as follows, from time of sowing to time of full maturity: November, 1.38 ins.; December, 3.27 ins.; January, 1.06 ins.; and February, 1.85 ins.; total, 7.56 ins. Irrigated on November 17th, December 3d and January 7th, each time with 40 cu. ft. per row; equivalent to 1¼ ins. of rainfall each time.

Notwithstanding this treatment, and the rainfall, they wilted considerably; some of the leaves died off, and they did not develop as they should. Irrigation was then commenced at the rate of 80 cu. ft. per row, equivalent to 2½ ins. of rainfall. Two of these irrigations were given, one toward the end of January and the other early in February, and the effect was very marked in increasing the growth and the color of the leaves.

Including the rainfall, the quantity of water was 16⅓ ins., which is

rather under the average summer rainfall of Maritzburg, with which tolerably fair maize crops are grown without irrigation.

The crop in question was fair, the average height being 5½ ft., but would have been better with earlier irrigation.

Another crop, under rainfall conditions alone (6 ins.), but sown later, has only attained an average height of 2½ ft., and is a failure.

Last year a poor crop was obtained on flat ground, and rather moist black soil, without irrigation, the total rainfall being about 14 ins. The writer is of the opinion that maize will benefit, and only obtain full development, with at least 36 ins., including rainfall.

Oats.—During the winter season, 1902, the writer's forage crop was decidedly good, and was grown entirely under rainfall conditions, but in the previous winter no crop was taken off the ground. Height, 2½ to 4 ft. Period of sowing, April 2d to June 4th. Period of reaping, October 6th to December 11th. Weight of crop per acre (straw and grain), about 1 ton. Rainfall for average period, 8¾ ins.

In this case there would be more moisture in the ground at the time of sowing than in the case of maize, as it was at the end of the monsoon, instead of the beginning, and besides the dews are heavy during winter and the evaporation less.

Potatoes.—During the season, 1901–02, the writer had a good crop of potatoes, for South Africa. These were not irrigated, as the rainfall was ample. They were planted on December 16th and were ripe at the end of March.

The rainfall during this period was as follows: December, 2.71 ins.; January, 3.79 ins.; February, 3.47 ins.; March, 4.99 ins.; total, 14.96 ins. The vines attained full development and flowered profusely, and some of the tubers were very large. The seed was Sutton's (imported) Early Rose. Yield, 3 tons per acre.

Lucerne.—The lucerne plots are 5 Gunter's chains in length by 22 ft., and in some cases 33 ft. wide, and are being watered by eight-arm sprinklers, working under a head of about 23 ft., at the rate of 1¼ ins. every 20 days, plus rainfall. It is too early yet to speak of the results with certainty, but there is a fair growth.

The writer, however, from his experience in the Cape Colony, believes that while 24 ins. per annum will produce a fair crop, 48 ins. are required for a good one, and the rainfall should be supplemented accordingly.

Apple Trees.—During the season, 1901–02, twenty apple trees produced 1 500 lbs. of fruit. The trees blossomed in September. The first apples were picked on December 16th, and the last on February 5th.

From the time of blossoming to the end of the fruit season the rainfall was 11.7 ins., and during this period each tree got three irrigations, each of 30 cu. ft.; a total of 90 cu. ft., which corresponds to, say, 2.3 ins. of rainfall when the trees are set about 22 x 22 ft. apart. The total quantity of water was therefore 14 ins. This was a good crop for South Africa.

During the season of 1902–03, the same number of apple trees only produced 900 lbs. of fruit. The trees blossomed in September. The first apples were picked on December 18th, and the last on February 18th. From the time of blossoming to the end of the fruit season the rainfall was 7.9 ins., and during this period the total irrigation, divided among all the trees, was 5 300 cu. ft., corresponding to 6.6 ins. The total quantity of water was therefore 14.5 ins. In this case, owing to the drought, there would have been no crop at all but for irrigation, as a tree higher up which could not be irrigated produced nothing although it had blossomed as well as the others. Owing to the scarcity, the apples sold at the Christmas market realized 4 d. per pound.

Lifting Water for Irrigation.—All the writer's irrigation has been done with ground-water. On the western boundary of the farm there is a sluit (stream) with a dry bed, except immediately after thunderstorms; and at one place on the bank there was a small spring which kept this particular spot moist. On this site the writer sank a well, lining it continuously with galvanized corrugated-iron sheeting, from 6 ft. above the ground surface to the bottom. The sheets were bent in the ordinary way in rolls, each to a semi-circle, as used for making tanks; but, instead of soldering them together, the writer fastened them with bolts and nuts, as it was not necessary or desirable that the lining should be water-tight. The total depth of the lining is 30 ft., the diameter at the top 8 ft. and at the bottom 6 ft. The lining was bolted up in sections, generally 4 to 8 ft. in height, and was passed down as sinking proceeded, the diameter, of course, diminishing continually from top to bottom. The top portion, 6 ft. above the ground level, was designed to keep out floods and cattle. The well was also roofed over to keep out leaves and branches from the neighboring

trees. A small door was placed on one side, for inspection, and a trap door in the flat roof to allow of additional sinking if required. A ladder was fixed inside to give access to the suction pipe and foot-valve.

The writer can recommend this method, in a country where brick or masonry lining is very expensive, until the water supply has been proved to be worthy of further expenditure, and afterward it would be easy to fix a mould and provide a thin concrete lining. An engine-house, 16 × 9 ft., of old corrugated sheets, has been erected just outside the well, in which is placed the lifting machinery, consisting of a Gardner oil engine, 1½ B. H.-P., and a Tangye double-acting horizontal pump, 4 ins. in diameter and with a 6-in. stroke. The suction pipe is 2 ins. in diameter, with one bend, and the average lift is 43 ft., to a large corrugated-iron tank, through 850 ft. of galvanized wrought-iron piping, also 2 ins. in diameter, with eight bends.

The main passes through lucerne beds, which are at the same level as the engine and pump, and is provided with crosses every 33 ft. having 1½-in. outlets closed by plugs, so that hose can be connected for attachment to sprinklers. The sprinklers usually work under the tank head, but occasionally they are worked under the pumping head when the engine is in operation. In the former case each sprinkler throws about 175 to 200 galls. per hour, but in the latter case the discharge is more than doubled, as they run very fast under the pumping head.

The pump throws 20 to 30 galls. per minute, depending on the pulley used, but could be run faster if necessary.

The engine, which works at 320 revolutions per minute, is also used to drive a corn mill which is placed alongside the pump, for grinding maize, wheat, beans, peas, etc., coarse or fine, as required for cattle and poultry feeding purposes, or for household use. With engine, pump, and piping new and in good order, the cost of lifting 1 000 galls. of water 43 ft. high through the main referred to is, for fuel alone, 1½ pence, that is for ¾ pint of "Homelight" kerosene oil at 11 shillings per case of 8⅓ Imperial gallons. This amounts to £1 14 shillings per acre-foot. In many cases 6 ins. of water plus the rainfall would make a good crop, and the writer thinks it would pay to lift water in this way, even with oil costing 20 shillings a case, which would include most districts in the Orange River and Transvaal Colonies which are served by railways.

A special and very careful measurement was made of the oil used, which includes both cylinder and lamp, and the water pumped was measured in the tank previously referred to.

Labor and Materials.

The labor question is a serious one in South Africa. The Kaffir is lazy, finds living very easy, and will not work continuously. The general public object to the importation of Indians on an extensive scale because they eventually enter into competition with small white traders, and occasionally with large ones, and, worst of all, the white man does not labor partly because he thinks it undignified while the Kaffir is here to do it for him, and partly because, as a rule, he is not offered a proper living wage.

If white labor is to develop the country, the cost of living, which is abnormally and improperly high, must be diminished. As things are, the mines and the various Colonial Governments monopolize all the colored labor for their works, and want more, and the unfortunate farmer comes in a bad third. With regard to materials, imported Portland cement is generally used for masonry and concrete on public works, the cost, in bond at Cape Town or Durban, for barrels of 400 lbs. gross, being about 11 to 13 shillings for good brands. The duty per 100 lbs. is 6 pence, say 13 to 15 shillings per barrel at the coast, duty free, and the cost of carriage by rail up country about 5 shillings per barrel per 100 miles.

This high charge and the long distances involved make the use of cement up country almost prohibitory, as far as economical works are concerned. The writer, in his "Report on Colonial Irrigation," advocated a thorough investigation into the nature and composition of the various limestones obtainable locally, with the view of ascertaining how far these might be substituted for cement in the construction of hydraulic works, but, as far as he knows, this has not yet been done in any of the colonies.

Stone fit for concrete can generally be obtained, but good clay for puddle is scarce in many parts of the country. Timber is dear, *viz.*, from 4 to 7 shillings per cubic foot at the coast, while the carriage by rail is fully 6 pence per cubic foot per 100 miles.

The Law.

The Common Law of South Africa pertaining to water rights is the Roman Dutch Law. This law divides streams into two main divi-

sions, *viz.*, public streams and private streams, and the former are defined as perennial streams.

Public streams are common to all for the purpose of supporting life, and therefore the rights of riparian proprietors are always limited by the natural rights of the public, as far as the latter are capable of being exercised. The Roman Dutch Law is sufficiently elastic to adapt itself to the special characteristics of the country. For instance, it has already been accepted as an axiom, in the Courts of the Cape Colony, that a river may become dry in the heat of summer without forfeiting its character as a perennial stream. That is to say, a distinction is drawn between streams which generally flow, and only occasionally become dry, and streams which are generally dry and only flow after rain.

It is therefore probable that the law is capable of adapting itself to practically all the needs of this country, as regards irrigation, in the same way as the Common Law in the United States has adapted itself already, in a great measure, to the needs of that country.

It does not follow, however, that the law is perfect as it stands; far from it. Some alterations are certainly required. The most urgent requirement is a right of passage of water act (carrying with it, of course, a right to expropriate land on payment of reasonable compensation), which can be utilized in the case of beneficent works, and, needless to say, governments strong enough to carry out its provisions, because the Cape Act, thus far, has been a dead letter.

If there is one thing which the Roman Dutch Law recognizes, it is that actual beneficent use must be proved before any right to water can be obtained, and this is a most important matter.

The Future Possibilities of Irrigation.

There is still ample room in South Africa for individual private enterprise to use ground-waters, pump from perennial rivers, and in some cases make reservoirs on large farms having private streams.

There is also room for enterprise on a large scale to lead water from streams too deep beneath the surface of the country to be dealt with by the ordinary farmer's furrow; but, in such cases, weirs, and other engineering works of magnitude, are required; however, until the law is amended, these works could not easily be undertaken by private enterprise.

It is possible, also, that a few large storage works might be carried out in the same way, but the writer is of opinion that, as a rule, it would not pay private enterprise to undertake them, as the cost of construction, under present circumstances, is likely to be very high.

It has been recommended by Sir W. Willcocks, of the Daira Sania Company, of Cairo, that all public streams in the Transvaal and Orange River Colonies should be proclaimed as state property, which, as the writer understands it, would result in all irrigation works being carried out by the Government for the good of the general public.

Thus far, this proclamation has not been made, but it is quite certain, on the other hand, that if irrigation is left wholly to private enterprise, the Government will be left, eventually, as in the United States, to deal with the storage problem.

Much can be said on both sides, but it must be remembered that in South Africa the Governments do not possess Crown Lands in the same wholesale way as America possesses Public Lands in the arid Western States.

In some parts of the Transvaal, particularly, there are ground-waters, on a very extensive scale, from which considerable development may be expected, and both in the Transvaal and Orange River Colonies most of the rivers flow for long distances with very little fall through the comparatively flat country of the central districts, and rather deep beneath the surface.

In his evidence before the Imperial Land Settlement Commission, at Cape Town in 1900, the writer advocated pumping schemes on a large scale as suitable for the country. It will therefore not be out of place to compare the existing conditions with those in Egypt, where pumping is largely resorted to and is successful.

In Egypt.—

1. English coal is imported and used successfully, the cost being about 45 shillings per ton;
2. Natives have charge of the machinery;
3. The lift is generally low;
4. The average winter temperature of Lower Egypt is 57° Fahr., and of Upper Egypt 60° Fahr.;
5. The mean annual rainfall of Lower Egypt is about 4½ ins., and Upper Egypt may be described as rainless;

6. The ruling price of wheat is 2s. 6d. to 3s. per bushel; maize, 2s. 3d. per bushel; beans, 2s. 9d. per bushel, and other produce on a similar scale;
7. The yield of maize varies from 35 to 50 bushels per acre (irrigated).

In South Africa.—

1. Colonial coal is available;
2. The Kaffir is far from being mentally inferior to the Egyptian;
3. The lift, generally, would be high;
4. The average winter temperature of Johannesburg is about 52° Fahr., which may be considered typical of a large portion of South Africa away from the coast, and at a considerable elevation;
5. The mean annual rainfall is much heavier than in Lower Egypt, but is erratic, as in all arid and semi-arid countries;
6. The price of wheat per bushel varies from 6 to 7 shillings at the coast to 7 to 9 shillings up country on the line of railway, maize 4 to 6 shillings per bushel, and other produce on a similar scale;
7. The yield of maize is about 16 bushels, without irrigation, but in certain districts and in favorable years it may be doubled.

In South Africa, as compared with Egypt, the disadvantages are temperature and lift, and the advantages, local coal, heavier rainfall, and higher prices for produce.

Three crops in the year can be obtained in Egypt, but, under the temperature conditions of South Africa, as a rule, only two, although in places like Maritzburg, Natal, three crops are also possible.

Coal is found over extensive areas in the Cape, the Transvaal and Natal, and in some cases the quality is very good. The price to the general public is unnaturally high, *viz.*, at the Cape about 22 shillings per short ton on trucks at the pit, although the Government gets it for the railways at about 15 shillings; but, doubtless, this will be remedied when things settle down. The cost of carriage varies from 2 to 4 shillings per ton per 100 miles, depending on the direction.

The writer is of the opinion that it would be advantageous to use windmills on a large scale in some parts of the country to lift water for irrigation; and Hydrographic District No. 12, in the Cape Colony, is a particularly suitable area. The following is taken from the records of the wind force at Kimberley, in that district, during 1899:

	Spring.	Summer.	Autumn.	Winter.
Number of days during which the wind force, during the whole 24 hours, exceeded 4 miles per hour	80	83	66	65
Number of days during which the wind force, during the whole 24 hours, exceeded 8 miles per hour	21	23	8	13

Ground-water is generally plentiful in this district at a reasonable depth from the surface. Sir W. Willcocks has estimated that in one-fifth of the Cape Colony, one-half of the Orange River Colony, and two-thirds of the Transvaal, maize, potatoes, roots, generally, and pumpkins can be grown with the aid of the rainfall, but the writer is somewhat inclined to think that these areas have been over-estimated, and the erratic nature of the precipitation and the farmer's difficulties under-estimated.

In some ways South Africa is not unlike the United States, for instance, the dividing line (20 ins. of rainfall) between the arid and sub-humid districts, may be roughly said to lie between Vryburg and Port Elizabeth (24th to 26th degree of longitude) cutting the country in half, while in the United States the same division also occurs about midway (97th to 100th degree of longitude), and in both cases the arid portion lies to the westward.

The principal difference lies in the fact that South Africa is more in need of irrigation than the United States, inasmuch as the western half is more arid and the eastern half less humid, making irrigation a practical necessity for successful agriculture over almost the whole of the country, with the exception of a narrow fringe along the coasts.

The following comparison with Spain, which is not on the whole a very thickly populated country, will give some idea of the area available for settlers.

TABLE No. 9.

Country.	Area, in square miles.	Population, including natives.	Date.
Spain.....	191 367	17 500 000	1890.
Cape Colony.....	277 151	1 500 000	1891
Orange River Colony.....	41 500	200 000	1890
Transvaal.....	119 128	750 000	1890
Natal.....	29 434	900 000	1898
South Africa.....	467 213	3 350 000	.....

The following distances are measured on the railways:

Cape Town to Bulawayo.....	1 322 miles.
East London to Bloemfontein.....	400 "
Durban to Johannesburg.....	483 "
Cape Town to Durban.....	1 463 "

In concluding, the writer hopes that his agricultural remarks and statistics may not be considered out of place, if somewhat unusual; for, if the hydraulic engineer is to cater for the irrigation needs of the farmer he should know something of the latter's crops, their seasons, requirements and value, and the difficulties which are encountered in growing them.

DISCUSSION.

Mr. Wright.

AUGUSTINE W. WRIGHT, M. Soc. C. E. (by letter).—The writer is pleased to note that the attention of this Society has been called to the agricultural point of view in irrigation, by Mr. Johnson's paper. As far as the writer's information extends, no papers before this Society, and hardly any engineering reports, dwell upon this subject from the point of view of the farmer.

In Southern California farmers are dependent upon sunlight, irrigation, and bacteria for all their fruits.

Considering these factors *seriatim*, the writer will first deal with sunlight.

Quoting from Professor King's book, "On The Soil," which it is hoped is familiar to all, he states:

"It is very essential that we firmly grasp the important part which both direct and altered sunshine play in plant growth; for while we have no control over the amount which may come to a given field, we can, and do, control the amount stored in the soil, and we also determine the number of plants which shall stand upon the field and utilize the sunshine which does come to it. * * * So powerful is the work of the sun against the cold ether of space, that Lord Kelvin estimates it at 133 000 H.-P. for each square meter at the sun's surface, and the working capacity of a cubic mile of sunshine, near the surface of the earth, he places at nearly 22 H.-P. Now, since the cubic miles of sunshine are arriving at the rate of 186 680 per second, each square foot of the earth's surface receives about one-seventh horse power."

Farmers are blessed with an abundance of sunlight in Southern California.

As to the question of irrigation, the writer is of the opinion that, as a rule, too much water is used in Southern California. All know that, of the two evils, too much water is worse than too little. It drowns the tree just as effectually as water would drown a human being. Hellriegel places the amount of water which should drain away from the soil, before it becomes habitable by cultivated plants, as high as 40 to 50% of their full capacity.

Professor King, speaking of the part played by water, says:

"Finally, with the advent of living forms upon our planet, water took up still another very important work; for to both plants and animals it is indispensable. Making up the larger part of their weight, it is the medium in which the chemical transformation, essential to the processes of growth takes place, if, indeed it does not play the part of an active and indispensable agent in bringing these changes about. Then, too, water not only takes up and holds in liquid form those substances in the soil which may become the food of plants, but it is a medium of transportation by which all materials are moved from root to leaf and from leaf back to the various places where the processes of growth are taking place. In the physiological processes of the animal body it has a similar and equally important role to play; for here, too,

Mr. Wright. it becomes first a solvent and then a medium of transportation by which all food is taken to and the waste removed from the various organs of the body.

"Finally, both in plants and minerals, water acts as a temperature regulator, tending by its evaporation from the surface to prevent the bodies becoming too warm, and among many animals this action is very marked."

Water being expensive in Southern California, for it is called worth $1 000 per miner's inch, or $100 per miner's inch per annum, the user is prone to take all he can get. The amount he can use beneficially is unknown to him.

Farming is becoming, day by day, more of a science, and the number of bright minds now engaged in solving its problems is greater than ever before in the history of the world.

From their investigations, the quantity of water required for various crops is now known. Professor King estimates the number of tons of water per ton of dry matter needed in Wisconsin for the following crops, as:

Dent corn....	309.8 tons.	Oats	522.4 tons.
Flint corn....	233.9 "	Field peas...	477.4 "
Red clover...	452.8 "	Potatoes	422.7 "
Barley	392.9 "		

Hellriegel found, through experiments conducted in Prussia, that the quantity of water withdrawn from the soil, and given to the air, almost wholly through the plant, was as follows, for each pound of dry matter produced:

Barley.....	310 lbs.	Horse beans.....	282 lbs.
Summer rye.....	353 "	Peas.....	273 "
Summer wheat..	338 "	Red clover.....	310 "
Oats	376 "	Buckwheat.....	363 "

This is at the rate of 325 tons of water for each ton of dry matter produced.

Proceeding to the consideration of bacteria, and quoting Professor King again:

"We come now to consider one of the most important discoveries of modern times, a discovery which places in the hands of every farmer a means, completely under his control, whereby he can at any time cause to be drawn directly from the atmosphere, the free nitrogen of the air, and have it fixed in the soil of any field he may wish to enrich. In 1888, Hellriegel published the results of his experiments, which satisfactorily proved the work in the soil done by bacteria."

Professor Cohn, writing of bacteria, but in relation to human life, stated:

"Way down below the limits of the human vision, exists the world of bacteria. So minute are these beings, that only high powers of the microscope reveal them, and they have been known a couple of centuries to exist, before there was any suspicion of their relation to our health. Only the last thirty years have disclosed their bearing on disease. * * * Their number is beyond conception, and their

powers of reproduction are scarcely credible; so great indeed are the latter that in many cases a single individual may produce sixteen million of offspring in twenty-four hours." Mr. Wright.

To day, thanks to the researches that have been made, the important part taken in all farming operations by bacteria is known. Given every other requirement, the farmer could grow no crop without the friendly co-operation of the minute bacteria. They enter into everything. All decay is due to their action. While some bacteria can endure several thousand degrees of heat, others can endure cold below freezing; but the greater number of the bacteria of decay are rendered inert at from 185 to 212° Fahr. If the article to be preserved be heated to the necessary temperature, and then sealed, so as to prevent the entrance of bacteria—ever present in the air—the article will be preserved for years. This is the foundation of all canning and preserving methods. These bacteria of decay are also rendered inert by a temperature approximating freezing, and this discovery has led to the investment of nearly $200 000 000 in cold-storage plants in the United States. The use of this process has saved the sheep interests of Australia from impending ruin, and the carcasses of their sheep are by this method transported to Europe, and sold at a price that enables many of the poor to indulge in the luxury of meat which, otherwise, they could not taste.

Captain Field, of the United States Navy, has told the writer that a steamer transports ninety thousand carcasses of sheep on one voyage! Figure the train capacity this represents!

The flavor of each variety of cheese is now known to differ, one from the other, because of bacteria. The varying flavor of tobacco is due to the same cause; and the Government, by introducing the bacteria of Cuba into the Connecticut River Valley is duplicating the flavor of the Cuban tobacco.

If one knew how much water the soil contained, he could measure the rainfall, and knowing the amount needed for his crop, he could put on the proper quantity, instead of "guessing" as at present.

Two years ago, the writer asked Professor Wiley, of the Agricultural Department at Washington, if the electrical method of soil moisture determination was applicable to the arid region. He replied that it had not been tested in the arid region, but, in his judgment, it was not applicable to those conditions. Had it been applicable, the writer would have used it, and thereby measuring the moisture in the soil would have had a better basis on which to estimate the quantity of water needed. At present, a miner's inch (12 960 galls. in 24 hours) is used for 8 acres of citrus fruits. Most of the fruit trees in Southern California are planted at 20 ft. from center to center each way—about one hundred and ten trees to the acre.

Of course, the field is graded before any trees are planted. As

Mr. Wright. pointed out by Professor Hilgard, the arid region really consists of three farms, one on top of the other. The writer has seen an excavation, 60 ft. in depth, with the same soil at the bottom as at the surface. In the grading for a ranch in Southern California, the hills are scraped down to fill the hollows. This condition is entirely different from that of the humid region. There, one encounters subsoil in which nothing will grow until it has been turned up and exposed to the "weathering" effects of wind, frost, and air for months. When a root reaches the subsoil in the humid region, its progress downward is stopped—it must spread laterally. Not so in the arid region. It is this difference in soil that permits the enormous crops to be grown in California. It is claimed that Alfalfa roots have been traced more than 100 ft. Its enormous spread of roots enables it to seek food, so that 24 000 lbs. are frequently cut from an acre within twelve months. The writer has known lemon trees to produce, within twelve months, 750 lbs. of fruit, or 8 250 lbs. per acre.

The irrigator prefers a good head of water, say 30 ins. or more, If the stream is small, and the soil at all sandy, the trees where he starts irrigating get more than their share, as the water sinks into the ground before the lower end of the field is reached.

Two systems of irrigation are in vogue. In one system, three or five furrows are plowed between each row of trees, and the water is turned into one furrow after the other until the field is irrigated.

In the other system, ridges are thrown up dividing the field into squares of 20 ft., and one square is filled, and then another, until the field has been gone over. It is obvious that this system allows better control of the water; but, on the other hand, if the water be allowed to stand in these compartments, diseases will develop in the tree.

"Now, the water placed upon the field may break away and run off; it may be lost by circulating downward beyond reach of root action, or it may be lost by surface evaporation from the ground itself. As a rule, it is only that water which really passes through the plant which contributes materially to its growth."

Recognizing this fact, it is customary to cultivate the field just as soon as it is dry enough, for it has been found that fine earth on the surface retains moisture in the soil in the best practicable manner.

It must not be forgotten that the soil must be pervious to the air, otherwise, nothing will grow. It is customary in Southern California to irrigate each three weeks during the dry season.

When one considers that, with a large crop, Southern California ships thirty thousand carloads of oranges and lemons alone in twelve months, on which she pays the railroads nearly $10 300 000 in freight, the importance of this industry will be realized; and the importance of irrigation to so many localities not now served.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The attempts at irrigation described by the author are certainly very crude, for a

thriving British Colony, and lead one to suppose that for this some climatic condition is responsible. Mr. Le Conte.

The first thing noticed is the extraordinary variation in the annual rainfall, namely, from 6 to 33 ins. In this respect, Cape Colony is somewhat like certain sections of California, and for sundry reasons it may be assumed that mammoth catchment reservoirs will become necessary, in the natural course of events, in order to tide over the lack of water during dry years, which seem to come in series. This fact alone, more than any others, adds enormously to the first cost of the proper development of an irrigation system.

It is unfortunate that so few run-off records are in existence, for they are the most reliable data upon which the engineer can depend. He should exercise unusual conservatism in selecting the minimum rainfall, and in assuming the percentage of probable catch, if he wants to get a reliable supply.

Table No. 10 contains the results of more than thirty years' actual experience in California, with storage reservoirs, and will be helpful in guiding the judgment.

From this table it will be readily seen that enormous storage room is absolutely necessary in order to insure a reliable supply of 1 000 000 galls. per day.

TABLE No. 10.

Average minimum annual rainfall, taking the three driest years in succession.	Number of square miles of watershed required to furnish daily 1 000 000 galls. during a severe drought.	Maximum storage room advisable for each square mile of tributary watershed, in millions of gallons.	Maximum storage room advisable for each million gallons per diem actually delivered, in millions of gallons.
San Francisco.			
No. 1: 40 ins.....	3.00	280	840
No. 2: 35 "	5.00	250	1 250
No. 3: 30 "	7.00	200	1 400
No. 4: 29 "	6.25	225	1 406
Oakland: 17 "	9.00	150	1 350

The estimated minimum run-off, of 5% of the average rainfall, in South Africa, speaks for itself, and illustrates the severe droughts to which the country is subject at times.

The author's remarks about the Roman Dutch Law, which prevails in the Colonies, are quite amusing. His statement that:

"If there is one thing which the Roman Dutch Law recognizes, it is that actual beneficent use must be proved before any right to water can be obtained."

Such a praiseworthy law as this would be a God-send to California, and would do away with 90% of the litigation now going on, and

Mr. Le Conte. which has brought forth decisions, first one way and then another, until the whole legal system is racked with doubts and uncertainties on every point.

The Common Law of England is in force in California, and this law will never fit the vital necessities of the community, and will have to be modified, sooner or later, to meet the demands of common sense.

It would seem, however, that, after all, the Colonial Government is naturally the proper party to undertake the construction of suitable irrigation works.

Mr. Johnson. FRANCIS ROBERT JOHNSON, M. Am. Soc. C. E. (by letter).—With reference to Mr. Wright's remarks, it must be acknowledged that farmers, everywhere, are dependent upon heat, light, air, water, plant food, and bacterial processes for their crops, but the writer's comparison between the winter temperature of part of Natal, and the summer temperature of part of Canada, shows the futility of heat without light, and, unfortunately for the farmer, it does not follow that, because from 376 to 522 tons of water are supplied, even to a fertile soil, he will reap a crop of, say, 1 ton of the straw and grain of oats (dry matter). This quantity of water, equivalent to about $3\frac{3}{4}$ to $5\frac{1}{4}$ ins. of rainfall, will not, under practical conditions, grow such a crop anywhere, for the simple reason that the plants cannot make use of all of it, however skilfully it may be applied; but, by applying a larger quantity, a crop might be able to make use of, say, $4\frac{1}{2}$ ins., and elaborate a ton of dry matter. In short, if the annual rainfall is reduced to $4\frac{1}{2}$ ins. falling in 3 or 4 months, where would farmers be found willing to risk their seed?

The factor of efficiency of the plant's machinery must be considered, and, unfortunately, this has received very little consideration from either scientists or cultivators. Owing to the advantage which it has of being within everyone's reach, probably the ordinary method of determining the percentage of moisture in the soil is sufficient, particularly considering the difficulty of estimating the depth within which any particular kind of crop can make use of this supply, or the degree of such use. Perhaps a mere statement of this percentage and of how it was obtained, before and after any agricultural experiments on water supply, would be better than any attempt at making an allowance. To ignore it altogether is certainly incorrect, and serious errors have thereby been made in the past, particularly when fallowing has been practised, or the land flooded in any way previous to sowing.

It seems to the writer that the direction in which to work, so as to benefit the irrigator as much as possible, would be:

First.—To ascertain for various classes of soil the minimum quantity of water which must be applied to produce an ordinary average crop, apart from what the plants actually use;

Second.—The maximum quantity from which a crop is capable of obtaining any practical advantage;

Third.—The best subdivision of the supply during the varying periods of growth. Mr. Johnson.

The writer thinks that cultivators might do a great deal for themselves in these directions by noting carefully the facts in connection with the growth of their crops. Even under rainfall conditions, much can be learned, particularly as regards the best subdivision of the supply.

While some attention has been given to the question of water supply, it is very doubtful if any worth speaking of has been given to the equally important questions of light and air to ordinary farm crops, particularly cereals, clovers, and grains; and it would appear that these questions should be considered in connection with any experiments on water supply. Light and air being quite as necessary as water to the vital processes, it follows that the quantity of the latter must depend to some extent on the supply of the former two.

In the case of crops sown broadcast or at narrow intervals, it cannot be said that either light or air is well distributed to the plants, while in both cases it is practically impossible to cultivate the ground after irrigation except in the very earliest stages of growth.

The writer, during his farming experience, was much impressed with the possibilities of what even poor soil will bring forth under favorable conditions, and was impressed still more with the difficulties (not wholly agricultural) which surround the cultivator—difficulties which in many places are "killing the goose that lays the golden eggs"—therefore any little help which hydraulic engineers can render him in making the most of his water supply, is in the right direction; and if piping or similar methods are to be used generally for distribution to ordinary farm crops, so as to minimize losses, the additional expense can only be met by a better knowledge as to when water is required, how little will answer the purpose, and the most efficient method of laying out the crop to meet not one, but all, of Nature's demands.

The writer is quite aware of the benefits which scientists have conferred on agriculturists, but thinks there is still room for many practical experiments, tending to increase the yield of ordinary farm crops, quite apart from expensive manuring.

In reply to Mr. Le Conte, who, apparently, refers more particularly to the Cape: Undoubtedly the annual rainfall varies very much in different parts of the colony, owing to the fact that it is subject to the influence of two separate monsoons at different seasons, and that there is a certain area influenced by both.

Tables Nos. 11, 12 and 13 will give a better idea of what these variations really amount to, apart from the seasons. Table No. 11 shows the greatest average "annual" variations within the various hydrograpical districts, while Table No. 12 shows the "annual" and "three-year period" variations, expressed as percentages, and the range, for various

Mr. Johnson. stations within the districts. Table No. 13, which is a summary from a paper,* "On Mean or Average Annual Rainfall, and the Fluctuations to which it is Subject," by Alexander R. Binnie, gives the percentages and the range for the average of thirteen stations in Russia, France, Austria, etc., having an average annual rainfall under 20 ins. (actually varying from 9.82 to 19.83 ins.). It will be seen that while the range at the Cape, as regards the yearly rainfall, is very much greater between 0 and 10 ins., as indeed might be expected under the circumstances, the range for the three-year period compares very favorably throughout.

Cost of materials and transport, rather than climate, are responsible for the paucity of irrigation works of any magnitude; indeed, the Public Work Minister of Natal has lately pointed out that it has cost more to transport produce from some of the remote portions of that colony to either of its chief towns, than to import to the latter from San Francisco.

Even where there are railways, the cost of transport is too high at present to allow of economical works in the back districts, and a considerable reduction is needed to encourage agriculture and irrigation, and thereby reduce the cost of living.

A small beginning has been made, with one of the larger rivers not far from Capetown, in the Robertson District. There, a number of farmers have combined to erect a weir, 1 200 ft. in length, with a maximum height of 9 ft., on the sandy bed of the Breede River, with the object of irrigating a large area of very fertile soil suitable for vineyards. This work has been completed, and is a marked success.

It must be remembered that the estimate of 5% of the average annual rainfall, as available for irrigation at the Cape, includes Great Bushman Land, which is one of the most arid regions on the earth, and the dry western districts.

With reference to the Roman Dutch law, perhaps the writer's remark was not quite clear. It referred to the rights acquired by prescription, on a private stream, to part or all of the water, as the case might be. Owing to the very large size of the original farms, most of which have since been divided and subdivided, the nature of the climate, and the few facilities for carrying out works enjoyed by the early settlers, the bulk of the present irrigation is from small private streams. On public streams, there is no right to the water, only a right to all owning land on its banks to use it, and all irrigators must abate equally during scarcity, and respect the ordinary rights of the public at all times.

On the ground that storage works interfere very much with existing conditions on any river, their construction by government agency would certainly appear desirable in the interests of the cultivators.

* *Minutes of Proceedings*, Inst. C. E., Vol. CIX, Part iii, Session of 1891-92, p. 89.

TABLE No. 11.—District Rainfall of the Cape Colony.

Hydrographic District.	Area, in square miles.	Total number of rainfall stations.	Average minimum, annual rainfall, in inches.	Average maximum, annual rainfall, in inches.	Average annual rainfall, in inches.
1.....	19 300	7	2.11	8.64	5.9
2.....	35 900	40	5.61	61.29	27.6
3.....	39 200	10	2.51	9.62	6.2
4.....	19 100	19	5.07	33.52	18.1
5.....	22 200	46	8.83	46.79	20.3
6.....	15 600	7	10.11	15.88	12.4
7.....	14 100	24	13.39	43.42	23.0
8.....	16 300	40	13.92	55.74	25.5
9.....	11 100	34	18.99	60.74	28.8
10.....	10 700	9	24.87	50.17	33.3
11.....	.....	Insufficient	data.	Insufficient	data.
12.....	.....	"	"	"	"

TABLE No. 12.—Station Rainfall of the Cape Colony.

Hydrographic District.	Rainfall station No.	Years observed.	Average annual rainfall, in inches.	Maximum annual rainfall. Percentage.	Minimum annual rainfall. Percentage.	Range. Percentage.	Average three wettest years. Percentage.	Average three driest years. Percentage.	Range. Percentage.
1.....	1	20	8.64	2.14	0.28	1.86	1.37	0.64	0.73
2.....	4	28	8.82	1.72	0.40	1.32	1.21	0.70	0.51
3.....	1	21	6.90	2.53	0.42	2.11	1.67	0.70	0.97
3.....	3	17	5.69	1.66	0.24	1.42	1.31	0.83	0.48
3.....	4	17	2.51	1.68	0.33	1.35	.....	.....	.....
4.....	2	21	9.27	2.05	0.48	1.57	1.53	0.63	0.90
4.....	3	20	9.24	1.76	0.34	1.42	1.31	0.68	0.63
5.....	3	29	8.83	1.75	0.35	1.40	1.40	0.73	0.67
6.....	4	18	10.11	1.69	0.39	1.30	1.46	0.66	0.80
7.....	2	21	16.17	1.69	0.51	1.18	1.39	0.78	0.61
8.....	7	20	15.31	1.58	0.40	1.18	1.30	0.73	0.57
9.....	7	17	20.59	1.58	0.34	1.24	1.37	0.71	0.66
10.....	2	15	26.02	1.43	0.52	0.96	1.30	0.75	0.55

TABLE No. 13.—Rainfall of Less than 20 Ins. in Various Countries.

	Percentages.					
	Maximum annual rainfall.	Minimum annual rainfall.	Range.	Average, three wettest years.	Average, three driest years.	Range.
Average of thirteen stations.	1.75	0.51	1.24	1.37	0.67	0.70

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 970.

DOCK IMPROVEMENTS AT LIVERPOOL.*

By George Cecil Kenyon, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Alfred Noble, L. J. Le Conte, Emile Low and George Cecil Kenyon.

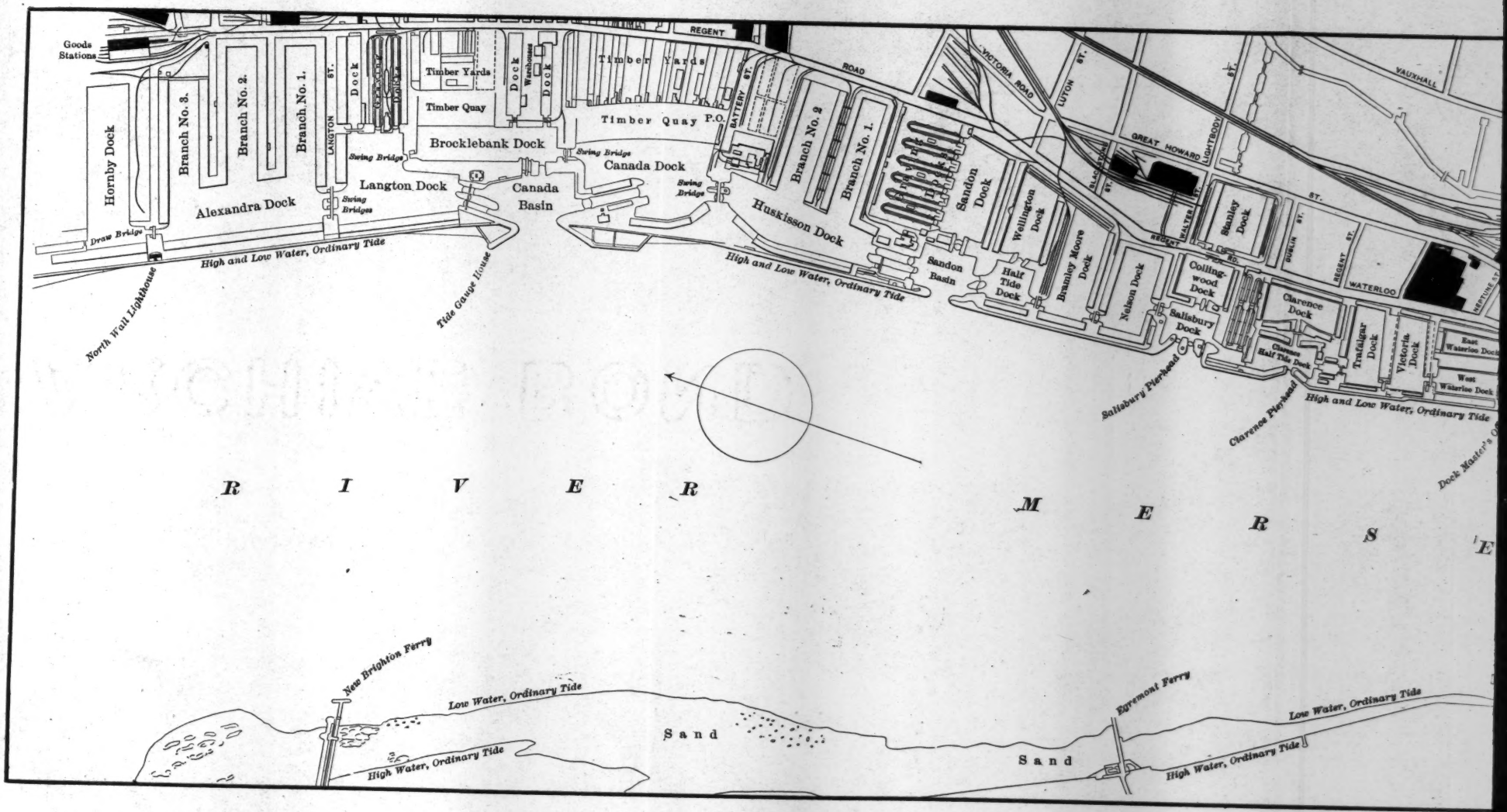
This paper, which describes the works now in progress in connection with the construction of new deep-water entrances at Brunswick Dock, Liverpool, has been written in the belief that a comparison of English and American methods of dock construction will be of interest to the membership of the American Society of Civil Engineers.

Anthony George Lyster, M. Inst. C. E., Engineer-in-Chief to the Mersey Docks and Harbour Board, is responsible for the design, which is being carried out under his direction, the writer acting as Resident Engineer.

As will be seen from Plate I, the Liverpool Dock Estate consists of two independent dock systems, the southern system being served by eleven entrances from the river.

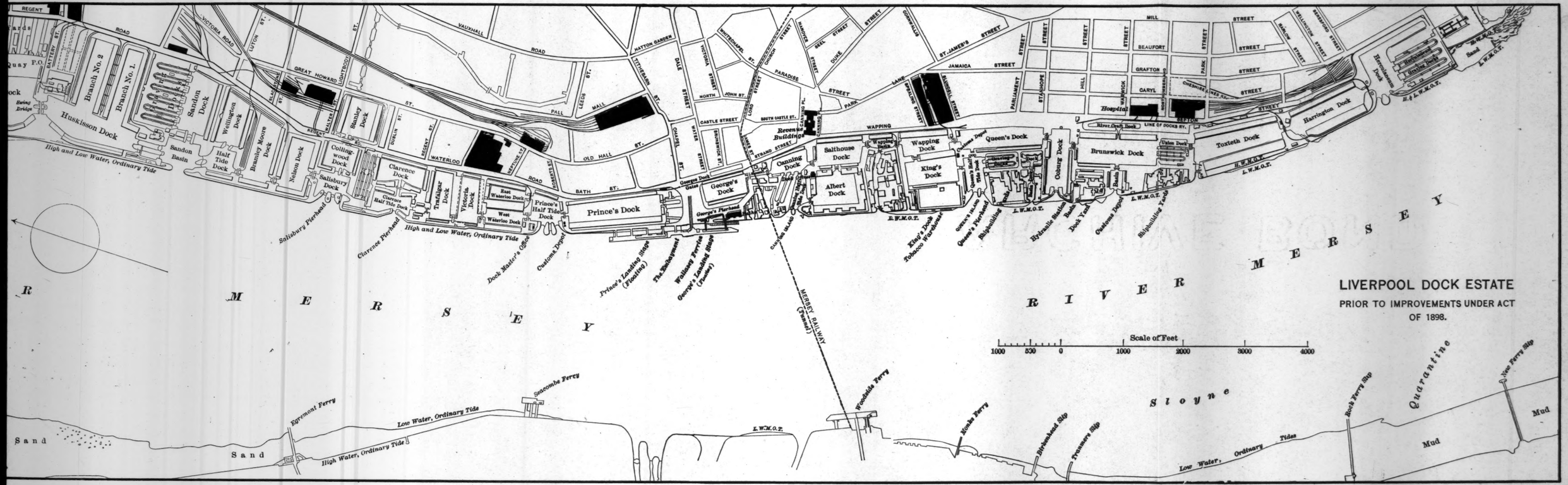
It having been decided to remodel the southern system by the construction of new deep-water docks and the deepening of some of the existing docks, new deep-water entrances became part of the scheme, as it will be obvious, by reference to Table No. 1, that the

* Presented at the meeting of November 18th, 1903.



Goods Stations
Hornby Dock
Branch No. 3.
Branch No. 2.
Branch No. 1.
Alexandra Dock
Draw Bridge
LANGTON ST.
Dock
Timber Yards
Timber Quay
Dock
Warehouses
Dock
Timber Yards
Timber Quay
P.O.
Brocklebank Dock
Swing Bridge
Langton Dock
Swing Bridges
Canada Basin
Swing Bridge
Canada Dock
Swing Bridge
BATTERY ST.
REGENT ROAD
Branch No. 2
Branch No. 1.
Huskisson Dock
Sandon Dock
Sandon Basin
Half Tide Dock
Wellington Dock
Bramley Moore Dock
Nelson Dock
Salisbury Dock
Collingwood Dock
Clarence Dock
Clarence Half Tide Dock
Trafalgar Dock
Victoria Dock
East Waterloo Dock
West Waterloo Dock
Stanley Dock
VICTORIA ROAD
LUTON ST.
GREAT HOWARD ST.
LIGHTBODY ST.
VAUXHALL
WALTER ST.
DUBLIN ST.
REGENT ST.
WATERLOO
NEPTUNE ST.
High and Low Water, Ordinary Tide
High and Low Water, Ordinary Tide
High and Low Water, Ordinary Tide
North Wall Lighthouse
Tide Gauge House
Salisbury Pierhead
Clarence Pierhead
Dock Master's O
R I V E R M E R S E
New Brighton Ferry
Low Water, Ordinary Tide
High Water, Ordinary Tide
Sand
Sand
Egremont Ferry
Low Water, Ordinary Tide
High Water, Ordinary Tide

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LII, No. 970.
KENYON ON LIVERPOOL DOCK
IMPROVEMENTS.



LIVERPOOL DOCK ESTATE

PRIOR TO IMPROVEMENTS UNDER ACT OF 1898.

existing river entrances, with the exception of the Herculaneum, are only suitable for comparatively small and shallow vessels, and are quite inadequate to provide means of access for the large type of cargo steamer now in vogue.

The scheme of reconstruction (see Plate II), for which an Act of Parliament was obtained in 1898, includes, in brief, the construction of deep-water entrances to the Brunswick Dock; the deepening of the Brunswick Dock and the underpinning of its walls in order that it may be used as a half-tide dock, or vestibule to the reconstructed system; the construction of a branch dock at the south end of the Brunswick Dock; the construction of a single-story shed on the east quay of the Brunswick Dock; the reconstruction of the passage between the Brunswick and Toxteth Docks; the widening and deepening of the passages between the Brunswick and Coburg Docks and the Coburg and Queen's Docks, respectively, and the deepening of the Coburg Dock between these two passages; the straightening of the north quay of the Coburg Dock; the deepening of the Queen's Dock and the construction on the west side of it of a graving dock and two branch docks with double-story sheds; the widening and deepening of the passage between the Queen's and Wapping Docks; and the construction of two branch docks with double-story sheds on the west side of the Wapping Dock.

The work done has been as follows: The east quay of the Brunswick Dock has been underpinned; the north quay of the Coburg Dock has been straightened; the passages between the Brunswick and Coburg Docks, and the Coburg and Queen's Docks have been widened and deepened, and the southern branch dock on the west side of the Queen's Dock has been completed.

At the present time, in addition to the new river entrances, the north branch dock and graving dock on the west side of the Queen's Dock are in course of construction.

In consequence of the last-mentioned works, the north and south entrances to the Queen's Half-Tide Dock, which have a width of 70 and 50 ft., respectively, and sills laid at 6 ft. 9 ins. below and copings at 31 ft. above datum, are permanently out of commission.

The river entrance to the Brunswick Half-Tide Dock has also been withdrawn from use, as far as the southern system of docks is concerned, as this dock is now used solely as a repairing dock by the

Mersey Docks and Harbour Board, and has no connecting passage with the Brunswick Dock.

At the completion of the works involved by the present scheme, the southern system of docks will be served by the entrances from the river, shown in Table No. 1.

TABLE No. 1.

Dock or Lock.	Width of entrance.	Sill below datum.	Copings of hollow quoins above datum.
Canning Half-Tide Dock, North	45 ft.	6 ft. 3 ins.	28 ft. 3 ins.
Canning Half-Tide Dock, South	45 "	6 " 3 "	28 " 3 "
Duke's Dock	40 "	4 " 2 "	25 " 10 "
Coburg Dock	70 "	5 " 7 "	30 " 6 "
Brunswick Lock (240 ft. long), North	80 "	19 " 0 "	33 " 0 "
Brunswick Lock (350 ft. long), South	100 "	19 " 0 "	33 " 0 "
Toxteth Lock (177 ft. long)	50 "	8 " 0 "	31 " 0 "
Harrington Lock (131 ft. long)	22 "	5 " 9 "	31 " 0 "
Herculaneum Dock, North	80 "	12 " 0 "	31 " 0 "
Herculaneum Dock, South	60 "	12 " 0 "	31 " 0 "

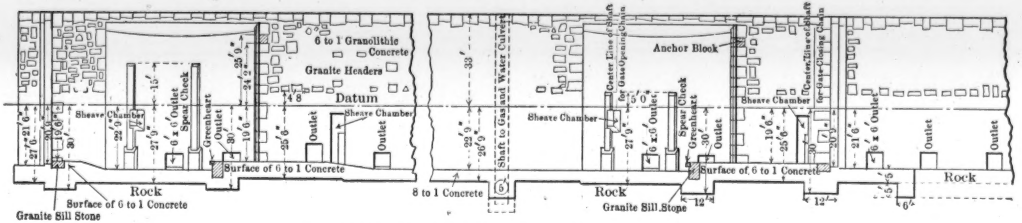
The new entrances have the additional advantage of allowing vessels to be passed to and from the docks for a much longer period during each tide than was formerly the case.

Any loss of water which may take place under the new conditions, through locking, etc., and on neap tides, will be made good by pumping from the river with three 56-in. centrifugal pumps, the pumping station being situated at the southwest corner of the Coburg Dock.

The new entrances, consisting of two locks, run in a northerly direction from the river and at an acute angle with the general line of the river wall. This is the direction now generally adopted at Liverpool as being the most suitable for passing vessels between the docks and the tideway.

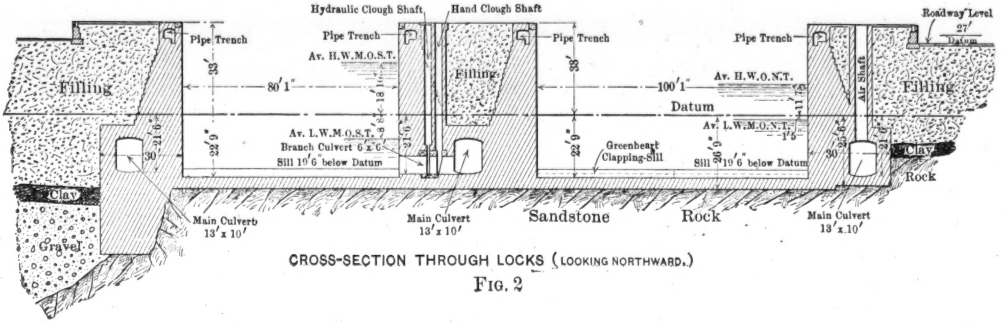
The eastern lock is 350 ft. long, between centres of hollow quoins, and 100 ft. 1 in. wide; and the western lock is 240 ft. long, between centres of hollow quoins, and 80 ft. 1 in. wide.

The level of the gate-sills in both locks is 19 ft. 6 ins. below datum, and as 6 ins. are deducted on account of the gate-closing chains, which cross the sills, the working depth is 19 ft. below datum. On comparing the working depth with the figures given in Table No. 1, it will be seen that there is a minimum depth of 9 ft. of water on the sills at low water of spring tides.



LONGITUDINAL SECTION OF THE 100-FOOT LOCK.

FIG. 1



CROSS-SECTION THROUGH LOCKS (LOOKING NORTHWARD.)

FIG. 2

The level of the coping is 33 ft. above datum, the lock floors and gate platforms being laid at a level of 22 ft. 9 ins. below datum.

The island between the locks varies in width from 65 ft. at the river end to 55 ft. at the dock end. Running longitudinally through the island and side walls there are levelling culverts, which will also serve for sluicing purposes. See Fig. 1, Plate III.

There are two pairs of gates to each lock, the operation of opening or closing any pair occupying about two minutes. The hydraulic power for gate machinery, etc., is obtained from the Boards' own mains, under a pressure of about 800 lbs. per square inch, the diameter of the pressure pipes being 6 ins. and of the return-water pipes 7 ins.

The gate recesses, Fig. 1, are corbelled over, the coping of the corbels being set to the same alignment as the walls of the locks.

At the back of and parallel to the side-walls are retaining walls (see Fig. 2), the intervening space being filled in so as to form a continuous terrace, 40 ft. wide, on the roadside of which are ornamental cast-iron railings.

These terraces have been formed with the object of giving greater facilities, in the docking of modern vessels, than would obtain if the copings had been constructed at the ordinary quay level, *i. e.* 27 ft. above datum.

Steel drawbridges give pedestrian access to and across the island.

The datum level referred to throughout this paper is 4 ft. 8 ins. below ordnance datum. The following are particulars of the average levels of certain tides, etc., with regard to this datum:

Average high-water mark of ordinary neap tides	11 ft.	7 ins.	above datum.
Average high-water mark of ordinary spring tides, excluding equinoctial tides	18 "	10 "	"	"
Average high-water mark of equinoctial spring tides	21 "	1 "	"	"
An extraordinary high tide, January 20th, 1863	23 "	9 "	"	"
An extraordinary high tide, February 13th, 1899	24 "	0 "	"	"
Mean tide level	4 "	8 "	"	"
Average low-water mark of ordinary neap tides	1 "	5 "	below datum.	

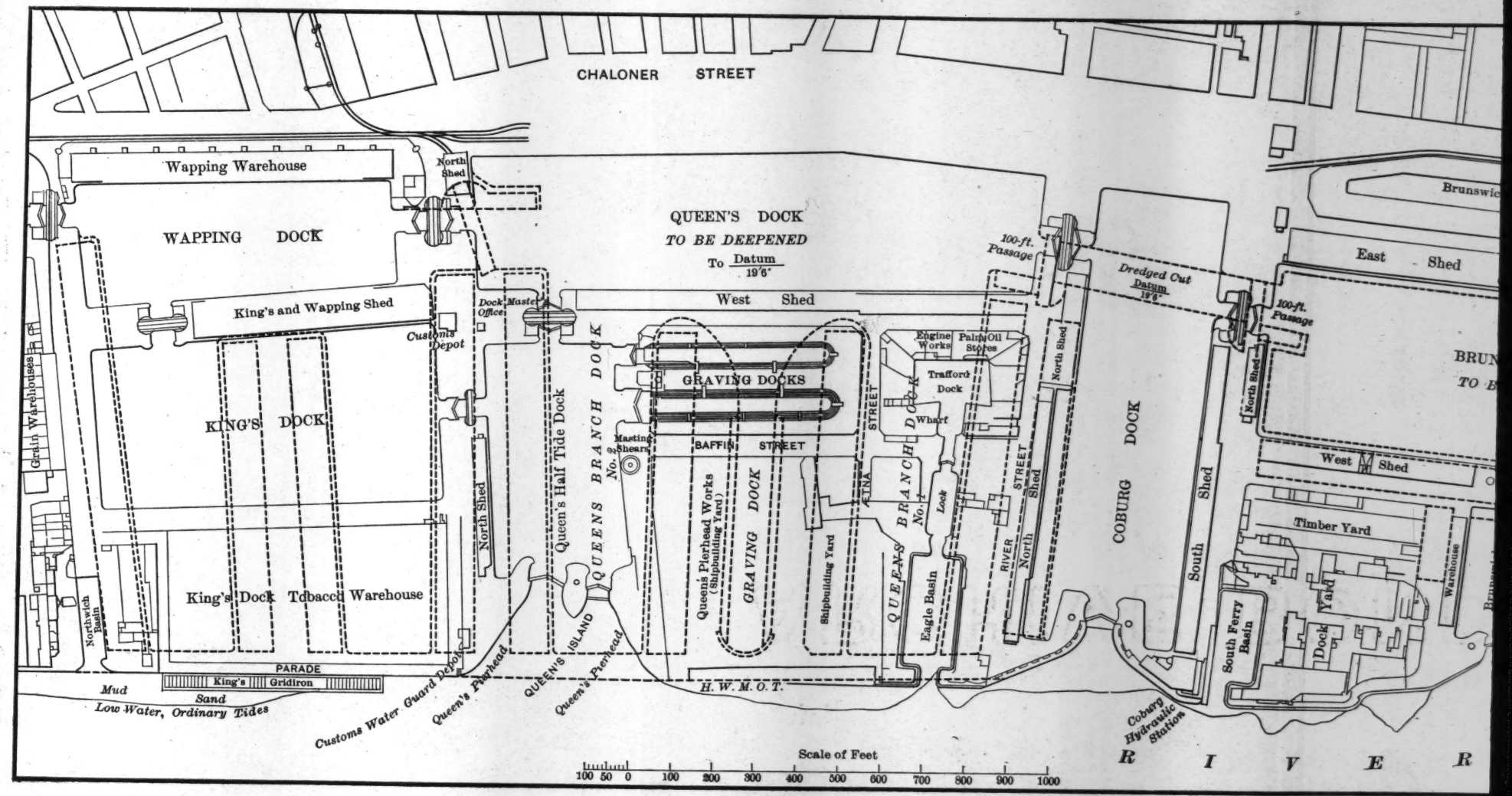
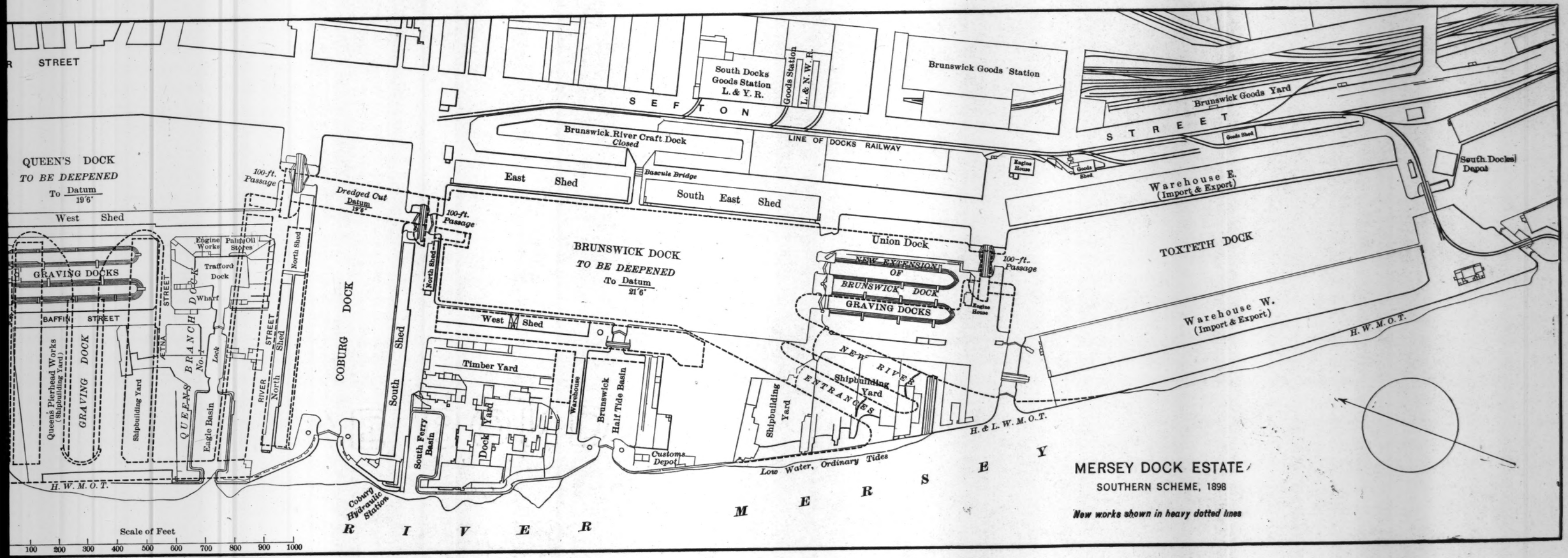

CHALONER STREET
Wapping Warehouse
North Shed
WAPPING DOCK
King's and Wapping Shed
Dock Master's Office
Customs Depot
KING'S DOCK
Grain Warehouses
North Shed
King's Dock Tobacco Warehouse
Northwich Basin
PARADE
King's Gridiron
Mud
Sand
Low Water, Ordinary Tides
Customs Water Guard Depot
Queen's Pierhead
QUEEN'S ISLAND
Queen's Pierhead
Queen's Half Tide Dock
QUEENS BRANCH DOCK No. 2
QUEEN'S DOCK
TO BE DEEPENED
To Datum 19'6"
West Shed
GRAVING DOCKS
Masting Shears
BAFFIN STREET
Queen's Pierhead Works (Shipbuilding Yard)
GRAVING DOCK
Shipbuilding Yard
ETNA STREET
QUEENS BRANCH DOCK No. 1
Lock
Eagle Basin
Engine Works
Palm Oil Stores
Trafford Dock
Wharf
100-ft. Passage
Dredged Cut Datum 19'6"
North Shed
RIVER STREET
North Shed
COBURG DOCK
South Shed
Coburg Hydraulic Station
H.W.M.O.T.
Brunswic
East Shed
100-ft. Passage
BRUN
TO B
North Shed
West Shed
Timber Yard
Warehouse
Dock Yard
South Ferry Basin
Scale of Feet
100 50 0 100 200 300 400 500 600 700 800 900 1000
R I V E R

PLATE II.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LII, No. 970.
KENYON ON LIVERPOOL DOCK
IMPROVEMENTS.



MERSEY DOCK ESTATE
SOUTHERN SCHEME, 1898

New works shown in heavy dotted lines

Average low-water mark of ordinary spring tides, excluding equinoctial tides..... 8 ft. 8 ins. below datum.
Average low-water mark of equinoctial spring tides.....10 " 0 " " "

It is not anticipated that during the construction of the entrances it will be necessary to run dry any of the adjoining docks for the purposes of, or in connection with, the works; the water being maintained at the ordinary working level.

It was known, however, that these docks might have to be run dry at some time during the progress of the work, for repairs to gates, walls, etc., and the possibility of this was borne in mind and provision made against all risks of damage arising therefrom.

During the progress of the work, when artificial light is required for any length of time, illumination is obtained by ordinary gas lamps, and in other cases by the Lucigen type of portable lamp.

The site of the new entrances had formerly been used for shipbuilding and repairing purposes, and the old river wall had been carried up to an average level of about 10 ft. above datum. There was also an old entrance in the wall leading to a small dock.

Before commencing the work proper it was necessary to raise temporarily the river wall (see Fig. 2, Plate III) to prevent the tide from flooding the works. The reconstruction of the northern portion of this wall was part of the scheme of improvement, and it was decided, therefore, to proceed with this work immediately. The line of the new wall being about 2 ft. in front of the old wall, the latter was faced with concrete to that extent, and the work of reconstruction completed by raising the whole in 8 to 1 concrete to the level of 26 ft. 6 ins. above datum. For the reconstruction of the river wall, a staging, forming a crane and wagon road, was erected about 2 ft. behind the line of the back of the new wall.

From the south end of this staging, a temporary railway was laid to the Brunswick Dock, where gravel, etc., for concrete, was discharged; a branch line was also laid to the cement stores.

To enable the faced portion of the reconstructed wall to be put in, a trench was sunk in front of the wall, and for pumping this out hand chain-pumps were used.

For timbering the trench, ordinary vertical poling boards with deal walings were used, the latter being strutted to the old wall.

For bonding the new concrete with the old wall, dovetailed pockets, 2 ft. wide at the face, 2 ft. 9 ins. wide at the back, and 1 ft. 3 ins. deep, were cut vertically in the old masonry, 4 ft. apart.

"Soldiers" and shutters were used in moulding the concrete at the face of the wall, and, at the back, the altar-courses were formed with "plums."

Simultaneously with the reconstruction of the river wall, the work of sinking the pump-well was in hand. Chain-pumps having four sets of reels, with buckets 2 ft. long, were used, the reels being arranged to work independently or together. The site selected for the pumps was at the northwest corner of the 80-ft. lock.

Anticipating that the general foundation level would be about 27 ft. 6 ins. below datum, the well was sunk to an additional depth of 10 ft., it being thought that this would be ample for all drainage purposes.

The engine and boilers for driving the pumps were placed in the rear of the trench for the west wall of the 80-ft. lock, at a sufficient distance beyond the angle of natural repose, taken from rock level, to prevent the possibility of the engine foundations being affected by any fretting of the ground caused by trenching operations.

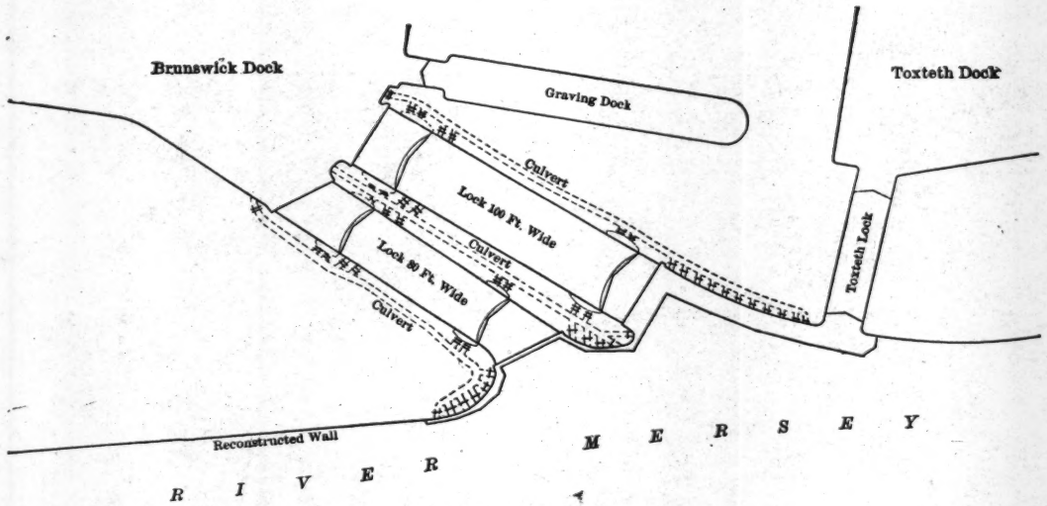
The power was transmitted to the reels by means of a shaft made up of sections with claw couplings, which allowed for settlement of the intermediate bearings without undue strain on the shaft.

The chain-pump well was sufficiently large to enable the whole of the framing carrying the reels, etc., to be put in before commencing the general excavation, it being intended that the original framing should remain without alteration until the completion of the work.

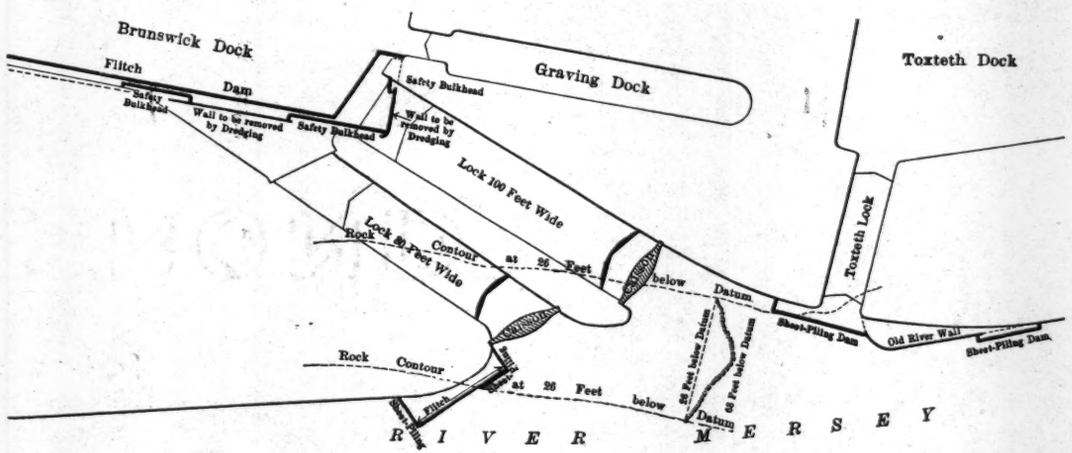
The intermediate bearings of the shafting were carried on needles, supported by piles braced longitudinally and laterally, those piles coming in the wake of the trench being underpinned and legged up as the work of trenching proceeded. The water from the pumps was discharged into the river, a distance of 380 ft.

From the south end of the reconstructed wall, and extending southward along the old wall, a making-up dam was formed of concrete blocks to raise the level to 24 ft. above datum. The wall was brought up, with 10 to 1 concrete, to a uniform level to form a foundation on which to lay the blocks.

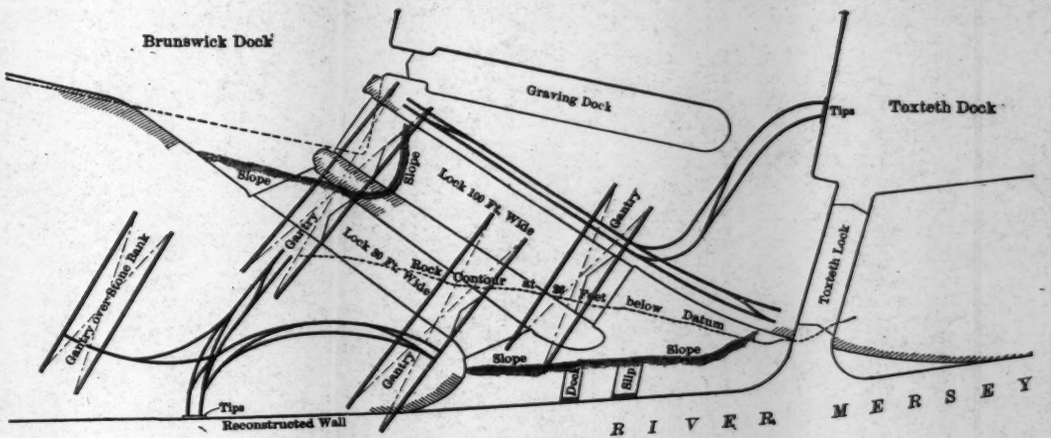
Blocks which had been used previously at other works were then



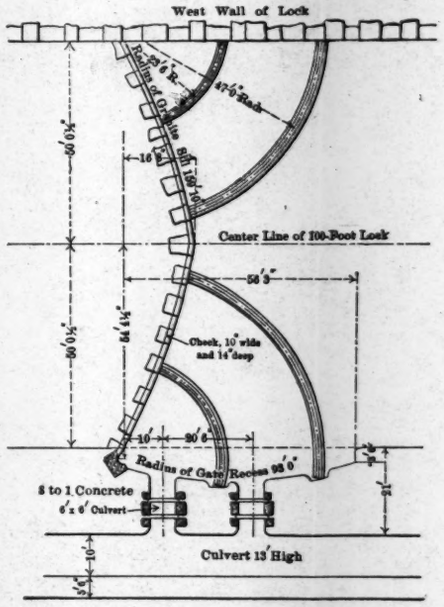
BRUNSWICK LOCKS.
GENERAL PLAN OF THE ENTRANCES, SHOWING CULVERTS.
FIG. 1.



BRUNSWICK LOCKS.
SYSTEM OF DAMS SUBSEQUENT TO REMOVAL OF RIVER WALL.
FIG. 3.



BRUNSWICK LOCKS.
THE HATCHED PORTIONS OF THE WALLS WERE NOT FOUNDED WHEN THE WATER WAS LET INTO THE WORKINGS.
FIG. 2.



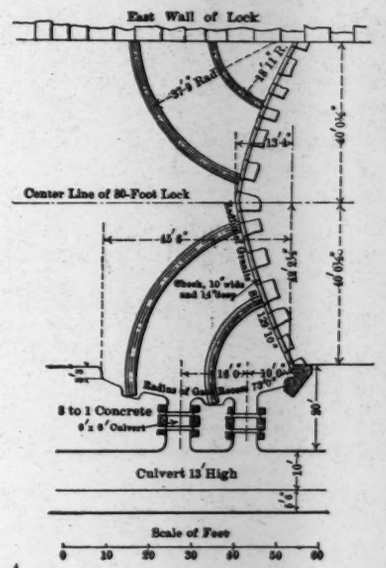


FIG. 4.

built up in courses, with mortar and brown-paper joints, and, when these were used up, other blocks were formed in position by placing shutters across the empty spaces between the headers and filling in with concrete, the bed and sides forming the moulds being well greased to prevent adhesion.

For setting the concrete blocks, a staging, forming a crane road, was erected at the back of the old wall. This road was also connected to the railroad leading to Brunswick Dock. The blocks were there discharged into wagons from flats and conveyed to the required positions by way of the above railroad.

The entrance to the small dock, previously referred to, was closed by horizontal whole-balk timbers, arranged to form a dam, the seams being caulked.

A timber dam was also placed in a gap forming an entrance to an old repairing slip, vertical deals, 3 ins. thick, with caulked seams, being spiked to walings spaced to suit the water pressure.

The method of disposing of the excavated material was by sending it to sea in the Board's steam hopper-barges. The distance, on the sailing line, to the dumping site is about 26 statute miles, the round trip occupying about 8 hours.

The hoppers loaded at the river tips sailed at each tide, those loading in dock at every alternate tide. End-tip wagons, holding about 4 cu. yds., were used in conveying the excavated material from the filling site to the loading berths, the wagons being hauled to the neighborhood of the tips by means of horses and tipped by "galloping out."

The east wall of the 100-ft. entrance and the west wall of the 80-ft. entrance were constructed in trenches, and, after their completion, the general excavation was removed and the island constructed. To minimize the risk of the trenches being flooded, it was considered advisable to terminate them at a distance of about 50 ft. from the old walls.

As anticipated, from borings taken prior to the commencement of the work, rock was found in the east trench about 12 ft. below datum, or 13 ft. 6 ins. above foundation level. The material was removed from this trench by means of skips, having a capacity of 2 cu. yds., which were raised by means of 5-ton steam-locomotive cranes working at both sides of the trench.

Parallel to the crane roads were full and empty wagon roads con-

nected by loops, these roads leading to a double tip-head at the Toxteth Dock, where the excavated material was loaded into hoppers. The roads were laid out as follows: Centers of crane roads, 5 ft. from the side of the trench; centers of near and far wagon roads, 16 ft. and 27 ft., respectively, from the side of the trench.

Owing to irregularities on the surface of the ground, and the limited space available for crane and wagon roads, the excavated material from the south end of the west trench was removed by means of steam jennies, the wagons being raised directly from the trench and run to the river tip-heads.

At the north end of this trench the excavated material was removed, as in the east trench, by means of cranes, the skips being tipped directly into hoppers berthed in the Brunswick Dock.

The trenches were commenced at a level of 24 ft. above datum, at which level the crane roads, etc., were laid, both trenches being 42 ft. wide over all, at the top setting of the deals. The shores and walings were of pitch pine, 1 ft. square, with walings 4 ft. apart, the struts being placed at 12-ft. centers. The deal runners were 3 ins. thick, and 18 ft. long, the top waling being placed at 1 ft. below the surface level.

The width of the trenches at the top was arranged so that, when rock was met at the expected level, 12 ft. below datum, there would be only about 1 ft. between the deals and the back of the wall at the base, the rock being sawn plumb with the back of the wall.

At the south end of the west trench, after excavating to a level of 30 ft. below datum, and rock not having been found, trial shafts were sunk, and rock was found at a level of 68 ft. below datum. Owing to this fault, which had not been disclosed by the borings, it was decided to found a portion of the wall, within the limits of the fault, on the gravel, at 30 ft. below datum, and to excavate to the greater depth for the foundations of the gate recesses and that portion of the wall south of the same, commencing 10 ft. north of the square quoin.

This involved a maximum lift of about 100 ft. for the excavated material, and, naturally, increased the cost of the work considerably.

It was apparent that the portion of the trench already commenced would become too narrow at the new foundation level, and, to obviate this, all runners then to be put in were placed a little out of the vertical, so that the same width was maintained down to the foundation level.

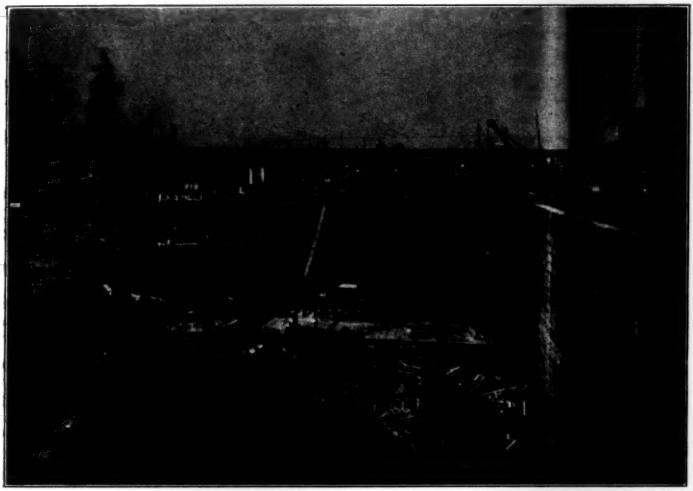


FIG. 1.—NEW ENTRANCE TO BRUNSWICK DOCK; 100-FOOT LOCK.

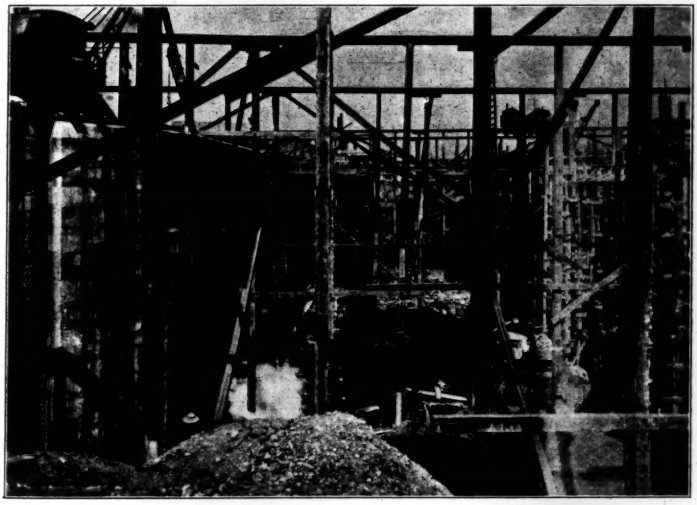


FIG. 2.—NEW ENTRANCE TO BRUNSWICK DOCK; 80-FOOT LOCK.

The extra depth to which the excavation had to be carried also necessitated the provision of an auxiliary pump, to throw water from the deep foundations to the level of the main pump-well. A 15-in. centrifugal pump was used for this purpose, steam being obtained from a boiler at the top of the trench.

With the exception of not finding rock at the expected level, there were no serious difficulties in carrying out the work in the west trench; but in the east trench this was not the case. As the line of the new east wall crossed the Toxteth Lock, the foundations of which were at a higher level than those of the wall, it was necessary to underpin the old work. For this purpose, when the trench had been excavated to the required depth, a heading was continued underneath the lock, the intention being to fill this in with concrete to correspond with the section of the new wall.

In driving this heading, an old constructional drain, 2 ft. in diameter, running longitudinally under the north wall of the lock, was encountered, the western termination of which was found to be in an apron outside the lock.

It was observed that the apron had parted from the river wall, causing an inlet to the drain, which, on being bared, burst, and, consequently, the entire length of the east trench and a small excavation which was being opened by a steam navvy were flooded.

As a safeguard against accidents of this description, a cross-bulkhead had been placed at the south end of the trench, but in spite of this the water found its way beyond the bulkhead.

Ultimately, the drain was plugged effectively by means of a canvas bolster, placed by a diver and filled with grout through a pipe from the surface.

The bolster was designed so that its total weight would be greater than the pressure of the highest tide on the sectional area of the drain.

The walls are set out to the curves shown at the coping, and are constructed plumb throughout their whole depth, and where joined to the old walls are thoroughly bonded, the faces of the walls being merged into each other. Between the new and old coping levels, ramps are formed.

The general section of the side-walls is 30 ft. at the base, this width being maintained to 4 ft. below datum, where it sets in to a width of

18 ft., from which it rises in altar-courses, 2 ft. high and 9 ins. wide, to a minimum width of 8 ft. 6 ins. The near side of the culvert is 14 ft. 6 ins. from the face of the walls, the inverts being laid at a level of 21 ft. 6 ins. below datum.

As regards the island, this is solid, with the exception of the culverts, to a level 4 ft. below datum, from which level the walls are constructed as in the side-walls, the space between the walls at the coping level being covered with concrete 1 ft. thick.

The lock floors are founded at 26 ft. 9 ins., and the gate-sills and caisson-sills at 25 ft. 6 ins. below datum. Under the gate- and caisson-sills, and projecting 3 ft. beyond the face of the same, are stoppers, 12 ft. wide, founded at 30 ft. below datum. The gate platforms are founded at 27 ft. 9 ins. below datum.

Above mean-tide level, the whole length of the new walls, with the exception of the gate recesses, retaining walls and reconstructed river walls, is faced with 1 ft. of 6 to 1 granolithic concrete, in which are set at intervals granite stones, with a 2-in. chisel-draft and rough-punched on the face, the sides being dressed to 3 ins. from the face. Such stones are not of less depth than 2 ft. at any part, and are thoroughly incorporated in the surrounding concrete, the aggregate surface of the headers forming not less than one-third of the total surface.

In the vertical faces the two qualities of concrete were placed in their proper respective positions simultaneously, and well trodden together.

The body of the wall and the floors of the chambers are of 8 to 1 concrete. In the back of the walls sheave chambers are formed of 6 to 1 concrete, 2 ft. thick, well worked around the bolts which attach the back plates of the sheaves to the wall.

The method adopted in constructing the east and west walls was as follows: The concrete was first brought up from foundation level to about 1 ft. below formation level, and the front of the walls was then formed by means of concrete moulds, the altar-courses at the back being moulded with deals. As the concrete was brought up, the shores and walings were relieved, the deal runners at the back of the wall being drawn after the filling had been put in to the height of a block setting.

After the moulds had been stripped, and before the shores could be

relieved, it was necessary to secure the timbering in front of the wall, that at the back being supported by the filling.

The concrete was raised in lifts of about 5 ft., two heights of moulds being used in this operation. Only one lift was made per week, the *modus operandi* being as follows: First and second days, mould setting; third day, filling moulds; fourth and fifth days, including Sunday, concrete setting; sixth day, stripping moulds; seventh day, relieving shores.

The walls of the culverts were moulded by means of horizontal deals, supported by deal cross-frames, 5 ft. apart, the concrete being put in evenly so as to obtain equal pressure on either side of the culvert.

The roofs were moulded by means of centers, supported by the verticals of the cross-frames, on which deals were laid to form lagging.

The inverts were formed in panels, in a hit-and-miss manner, after the arching over was completed, the radius being obtained by screeds.

The finished culverts are 10 ft. wide and 13 ft. high, the inlets being 7 ft. 6 ins. wide and 10 ft. high.

Circular air-shafts, 5 ft. in diameter, are brought to the surface in square pillars, 8 x 8 ft., and closed at the top by means of gratings.

Leveling culverts, 6 x 6 ft., connected with the main culverts, are provided in each lock, and constructed with double cloughs, one of which is worked by hydraulic, and the other by hand power.

The outlets of the culverts were moulded with deals, as in the main culverts, the sills, jambs and head for the cloughs having been set previously.

An iron-lined culvert, 5 ft. in diameter, for gas and water pipes, is laid under the entrances, and is connected with the surface, at its ends and on the island, by shafts of the same diameter which pass through the pipe trenches at the top of the walls.

Checks are formed in the gate recesses and side-walls for sheave chambers, gate chains, trucks and spears.

In the top of the walls, 2 ft. 6 ins. from the face line, a trench, 4 ft. 6 ins. wide and 5 ft. high, with arched roof 3 ft. thick, is formed for hydraulic pipes, gas and water mains, etc., the pipes being laid on a benching, 3 ft. 6 ins. wide, raised 1 ft. above the floor. Manholes are placed in certain positions, and openings left for hydrants, etc.

Cast-iron curbs and covers are provided at all machinery pits, air-shafts, clough-pits, etc. Lamp-posts, mooring-posts, mushrooms,

guard-posts and chains are also provided, and the roadways adjoining the retaining walls are to be paved with granite setts.

In front of and connected with the river portion of the new walls, there is an apron, 30 ft. wide and 5 ft. thick, with a stop sunk into the rock, 6 ft. wide and 5 ft. deep, at the outer edge.

In the east wall the concrete was hand-mixed, the gravel being discharged at the northwest corner of the Toxteth Dock, whence it was conveyed in wagons to the mixing stages.

In the west wall the concrete was machine-mixed, the gravel being discharged at the river wall and conveyed in wagons to the required positions.

All concrete consists of approved aggregates of gravel, broken bricks, or stones, with a proper proportion of sand, mixed with Portland cement of the best quality, as follows: 8 to 1, 6 to 1, and 4 to 1 concrete consisting of 8 parts, 6 parts and 4 parts, respectively, of aggregates, as described, to one part of Portland cement by volume.

"Plums" were placed in the body of the concrete, and inserted so that all parts of each stone were well surrounded by concrete, no stone coming nearer to the surface of the wall that 1 ft. "Plums" were set in the top layers of concrete while it was moist, and were placed so as to project into the next layer to form a bond.

The bulk of the "plums" was not included in the proportion of aggregates above mentioned, and all stones used as "plums" were of a specially hard and durable character, thoroughly sound and shapely, and clean of all mortar and foreign matter.

The cement was delivered long enough before using to give ample opportunity for making the tests hereinafter described. A watertight, dry, wooden-floored cement store was provided on the site, and the cement was delivered there in lots of not less than 100 tons; the lots being kept clear and distinct from each other, and the stock not allowed to fall below a month's requirements.

All cement used was of the best quality, free from any admixture of Kentish rag, gypsum, or other foreign material, the contractor being required to supply with each cargo a signed declaration from the manufacturer to this effect.

Samples were taken from each lot, after delivery on the site, and these were subjected to the following tests:

"Each sample, on being sifted through a No. 50 wire-gauge sieve, must not leave a residue of more than 5 per cent.

"The time of setting of the cement must not be less than 3 hours nor more than 4½ hours after mixing, each sample of cement being considered set when it no longer takes an impression from a Vicat's needle having a total weight of 2½ lbs. standing vertically on the cement. Other samples gauged neat will be rammed by hand pressure into moulds, and some of the casts thus made will, within 24 hours, be immersed in water and at the expiration of 7 days from the day of moulding, exclusive of that day, will be tested. Other casts made as above will be allowed to air-dry, and at the expiration of 7 days as above will be tested.

"Each of the casts will be tested in a De Michele machine, and must be capable of sustaining a tensile strain of not less than 450 lbs. on a sectional area of 1 sq. in."

The specific gravity of the cement as delivered was not to have been less than 3.1, and no cement from any lot was used in the works until such lot had been tested and proved satisfactory.

Any cement from which samples had been taken and had failed to stand any of the tests before mentioned, or which at any time before use had given signs of being unsatisfactory, was rejected, and, unless otherwise directed by the Engineer, was removed, at the expense of the contractor, and did not form part of any subsequent delivery.

The foregoing tensile tests, as far as possible, were made within 14 days of the full delivery of each consignment, and the results of the tensile tests of quantities rejected, owing to failure to sustain the required strain, were forthwith communicated to the contractor.

The gravel was supplied by contract from an approved source, and was delivered alongside the quays. It was sea-washed, clean, and free from clay, silt and other foreign matter, fairly coarse, free from boulders and fine sand, and no soft, inferior material, no brick-powder, dust or dirt of any kind was used in the concrete.

Strong gauges were provided for the cement and gravel, or other aggregate, to ensure that the proper proportions as specified were maintained.

In mixing concrete by hand suitable platforms were laid down and the mixing was done as follows. As the gravel gauge was being filled the cement was added to it gradually so that when the gravel gauge was full, the cement gauge was empty. The gauge was then removed and the heap leveled off to a uniform thickness of not more than 1 ft. and was then mixed by casting backward and forward twice, the water being added as the mixture was being cast backward the second time.

After these operations had taken place, the concrete was sent down shoots into position, care being taken to insure that no concrete should be put in unless its component parts were thoroughly mixed and incorporated. In windy weather screens were provided to prevent loss of cement.

The water used for the concrete, and for all other purposes on the works, was obtained from the city mains. It was applied carefully to the concrete from a rose in such volume as not to wash the cement from the gravel. No concrete was allowed to stand after being mixed, but was used in the work immediately.

The concrete work, as far as possible, was carried on continuously, layer by layer, so that the whole formed one homogeneous mass, care being taken to remove all scum arising from the cement before putting on another layer.

The surfaces of all masonry or concrete, on or against which concrete was to be laid, were thoroughly cleaned and wetted immediately before concrete was laid on, and all wooden moulds or containing boards were payed with composition to prevent the concrete from adhering to them. In face work all the concrete was well worked against the moulds when moist, the surface, when finished, being perfectly smooth.

No concrete work was commenced or proceeded with during frosty weather, except in approved positions and subject to certain conditions.

The granite for the hollow quoins, sills, cloughs, copings, etc., was obtained from the Board's quarry at Kirkmabreck, Kirkcudbrightshire, N. B., being brought from the quarries by sea, in the rough, and dressed on the site.

The dimensions of the copings are within the following limits: Not less than 1 ft. 6 ins. nor more than 2 ft. 6 ins. deep and of a similar width, the sectional area at right angles to the line of coping being not less than $3\frac{3}{4}$ sq. ft. and not more than $4\frac{3}{4}$ sq. ft., and the length of any stone not less than 2 ft. The stones have a rounded nose of 6 ins. radius, the top rising $\frac{1}{4}$ in. per foot from the front draft.

Each stone was chisel-drafted 2 ins. wide on the face, top and bottom beds and joints, punched down to drafts on the face and top bed and below drafts on the bottom bed and joints.

The back of the stone was not squared to the joints, but was dressed to 3 ins. below the top bed, where backed by concrete.

One foot back from the face the stones were joggled, throughout their whole depth, so as to form, with the similar groove in the adjoining stone, a 3-in. square hole, this being run with cement and selected gravel, and finished at the surface with a granite plug.

The stones were bedded solidly on the concrete beneath, and the joints flushed up with cement mortar, grouted and neatly pointed.

Cement mortar for all works was generally composed of 1 part of Portland cement to 2 parts of clean sharp sand.

The surface of the quay, behind the granite coping, was carefully finished with a 6-in. layer of 4 to 1 concrete.

Between the outer and inner caisson-sills the rock under the floors of the locks is drained by vertical pipes, 4 ins. in diameter, filled with pebbles and placed 5 ft. apart laterally and longitudinally.

The gate-sills of the entrances are formed of 8 to 1 concrete with a surface of 6 to 1 concrete 1 ft. thick.

The gate clapping-sills are curved in plan, and are formed by granite stones, 2 ft. 6 ins. to 3 ft. 6 ins. long, 2 ft. wide, not less than 4 ft. deep, and at 5-ft. centers, with 6 to 1 concrete between them.

The sill stones and concrete are checked to receive the sill wood, which is bolted thereto.

The face and the bottom of the check are axed, the remainder of the face and the top bed having a 2-in. chisel-draft all around, and being punched below the draft. The sides and back of the stones are drafted and punched to a depth of 6 ins.

The cast-iron roller paths for the gate-trucks (see Fig. 4, Plate III) are fixed in the following manner: The concrete is carefully brought up to within 1 ft. of the general surface, pockets being left for the holding-down bolts. The segments of the roller paths are then placed in position on brick piers, with the holding-down bolts hanging from them, the space underneath the segments being filled with 4 to 1 concrete, well rammed,

The hollow quoins are of granite throughout, except the anchor blocks, which are of cast iron. The quoin stones, which were dressed to an iron template, are arranged so that every other is a full quoin stone taking in all the curve, the alternate one being a full-necked stone, but not taking in the full curve, the remaining portion of the curve being formed of concrete moulded in position.

The stones are of irregular form, but each stone has a thickness of at least 2 ft. back from the hollow, and the total net content of each

is not less than 35 cu. ft. The top and bottom beds are fine-axed 6 ins. back from the face, the remainder of the bed being drafted on the edges, and punched flush with the joint surface. The face, in large and small rounds, is fine-axed all over, the hollow being fine patent-axed. The faced sides of stones are chisel-drafted vertically, 2 ins. chisel-drafted at the edges and rough-punched 6 ins. back from the inside face. All stones are set as carefully as possible, and, after the completion of the quoins, the necks are re-dressed and made plumb and true throughout.

In the top and bottom beds, grooves are cut, so that, with the similar grooves in the adjoining stone, a 3-in. square joggle is formed at right angles to the side-walls of the entrances; small grooves, also, are provided, to form joggles 1½ ins. square, to allow of the running of grout.

The joints between the stones are formed on the face by a 4-in. strip of 7-lb. lead. Behind the lead the stones are carefully bedded in neat cement and afterward grouted.

The heel-post stones, which receive the pivot cups, are not less than 30 ft. in superficial area, nor less than 3 ft. 6 ins. deep, and are set on a level, bed-punched all over.

The top surface is chisel-drafted at the edges and plain-axed all over, the top portion being axed very fine.

The heel-post stones are set at 1 ft. 10¾ ins. below the sill level, the respective centers for the 100-ft. and 80-ft. gates being 2 ft. 4 ins. and 2 ft. 2 ins. back from the face of the walls. The necessary square sockets to receive the cup castings were cut after the stones were set. A cast-iron block, 1 ft. 7 ins. deep, to fit the head of the heel-post, forms part of each hollow quoin. This block is bolted to the granite beneath by three 2-in. lewis-bolts, and is anchored to the mass of concrete behind by four 3½-in. wrought-iron anchor rods, fitted and wedged in the anchor block with a square head, the other end passing through a large block of granite, 6 x 3 x 2 ft., and having a wrought-iron washer 1 ft. in diameter and 1 in. thick.

Galvanized wrought-iron collar straps, 6 x 2½ ins., are provided for each heel-post, the straps being bolted to the block by two 3¼-in. T-headed, octagonal bolts.

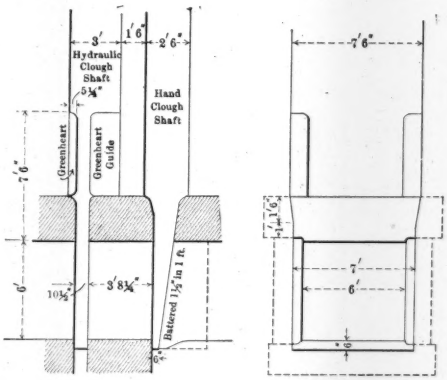
The square quoins, which have a bull-nose of 1 ft. radius, are formed in concrete to a level of 5 ft. above datum, from which level they are formed alternately in granite and concrete in 2-ft. layers.

The stones, which are set alternately as headers and stretchers, are not less than 5 ft. long and 3 ft. wide, having an area on bed of not less than 15 sq. ft.

Each stone is chisel-drafted 2 ins. wide on the face, the top and bottom beds being close-punched down to the drafts. Joints are chisel-drafted and punched down for 6 ins. back from the face.

All exposed surfaces of jambs, sills and heads of cloughs (see Fig. 3) are chisel-drafted and punched down below draft. The faces of grooves are polished, for 6 ins. back on the sills and jambs, and for not less than 10 ins. on the head stones.

The sills and heads are mitered to the jambs for a distance of 2 ins., with neat-cement joints.



DOUBLE CLOUGH SHAFTS FOR 6 X 6-FT. LEVELING CULVERTS.

FIG. 3.

The caisson-sills and quoins are of concrete and granite, no stone being less than 5 x 3 x 2 ft., and are dressed in a manner similar to the hollow quoins, with bearing faces fine-axed. They are set alternately as headers and stretchers, 3 ft. apart, the intervening spaces being filled with 4 to 1 concrete.

The general excavation was carried on both day and night, and proceeded simultaneously with the trenching. A sufficient distance was kept from the trenches so as not to interfere with the equilibrium of the timbering of the latter.

A Whittaker, 10-ton, steam navvy was used on this work, the wagons being lifted from the excavation to the tip levels by means of steam jennies arranged so as to serve also for erecting the lock gates.

Three lifts of 16 ft. each were taken by the steam navvy. In the first lift the material consisted principally of sand and rock dumpings; in the second, sand and gravel; and in the third, roach and rock. The bucket of the navvy had a capacity of about 1½ cu. yds. and filled

on an average two hundred and fifty 4-yd. wagons a day. For the satisfactory working of the navvy, two wagon roads, connected by jump crossings, were laid, one for empty and the other for full wagons.

Both dock and river tips were used, the former continuously: but, owing to the rise and fall of the tide, tipping could only be done at the latter during two hours before and two hours after high water. At the river, tip wagons were tipped at the rate of one every two minutes, representing about 480 cu. yds. of excavation for each tide.

The maximum quantity of excavated material sent to sea in any one week was 12 000 cu. yds.

All excavated material used for filling and for the formation of quays, roadways, etc., was deposited in horizontal layers of no greater depth than 1 ft., well watered and rammed.

Blasting operations were carried out to some extent, the explosives used being tonite and black powder.

In excavating for the island, the fault previously referred to was found to extend across the site of the south end of the island and of the outer caisson- and gate- sills of the 80-ft. entrance. In this case the excavation was carried down to 26 ft. 6 ins. below datum, from which level a trench 12 ft. wide was sunk to the full depth, the trench extending 3 ft. in front of the face line of the island and 9 ft. behind it. This trench was filled in with 8 to 1 concrete, the remaining material in the fault being undisturbed.

The foundations for the caisson- and gate- sills were laid in a similar way.

After removing the general excavation, to enable the south end of the island to be put in, a staging was erected along its center line, the height of decking being about 4 ft. below the coping level. On this were laid three lines of rails, the outside lines being used as crane roads and the center line for wagons.

The width of the decking was 40 ft., over all, and the cranes remained in position to set the coping. Concrete mixers were placed every 60 ft. along this staging, the wagon road being placed a little out of center to allow of this being done.

The gravel for concrete was discharged into side-tip wagons from vessels berthed in the Brunswick Dock; cement, "burrs" and "plums" were also brought to the required positions in wagons.

Through having no timber to relieve, the construction of the island was very rapid, two 5-ft. lifts of concrete being made per week, the moulds for the facings being supported by "soldiers" secured to the uprights of the staging.

The excavation for the northern portion of the island was then commenced, the excavated material being hand-filled into skips, lifted by the cranes on the staging, tipped into end-tip wagons, and used for filling between the walls of the southern portion of the island. The staging was removed as the filling proceeded, the uprights being sawn off flush with the concrete.

When sufficient excavation had been removed at the northern end to allow of the erection of another bay of staging, the work of excavating was stopped temporarily, the timber from the south end of the staging being used for the northern continuation.

The staging will be extended in this manner to the northern termination of the island, and, when the excavation is completed, the remainder of the island will be constructed in a manner similar to the southern portion.

After the southern portion of the island had been completed the outer gate- and caisson- sills, etc., were put in.

The gates, which had been constructed in the Board's shops, were then erected, after which the water was let in, to enable the river wall to be removed, the outer caissons to be placed, and the system of dams to be completed. (See Fig. 3, Plate III.)

By having the outer gates in position, any failure which might occur in the dams at the north end of the works would not endanger the shipping in the docks.

As much of the river wall as possible is being removed by tide-work, and the remainder by means of bucket dredgers. While the removal of this wall is in progress, the work inside the locks is, of course, stopped temporarily.

The junctions between the new and old walls at the north end of the entrances will be made and the old walls removed behind dams, the shores being relieved from the old to the new walls as the former are demolished.

It can be readily understood that, under ordinary conditions, the work might be delayed through "blows" under the dam in the Brunswick Dock while the northern portion of the work is in hand.

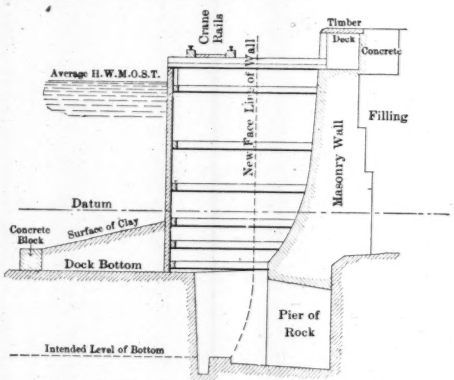
To minimize the risk of delay from this cause, the work between the dams and the wall was completed before breaking through the walls of the Brunswick Dock; safety bulkheads, founded on the new concrete, being constructed without interfering with the outer dams.

The Brunswick Dock is being deepened from 9 ft. to 21 ft. 6 ins. below datum, and the walls underpinned by adding about 12 ft. of concrete to their base, the foundation of the walls and the floor of the dock being in red sandstone of fairly good consistency and soundness.

The deepening of the floor of the dock is being carried out by dredging, and the underpinning behind flitch-dams, divided longitudinally by gussets to the wall at intervals of between 300 and 350 ft.; each section thus formed being drained by a set of chain-pumps erected upon needles supported by the quay and the dam.

The excavated material for the underpinning is removed in skips by 5-ton locomotive cranes, running upon crane roads mounted upon needles supported by the dam and the wall.

The space enclosed by the dams (see Fig. 4) having been cleared of water, a grip, about 8 ft. wide, is sunk in the rock immediately in front of the toe of the wall, to the required depth of underpinning, and on the dock side of this grip a further channel is excavated to convey drainage to the pumps.



METHOD OF UNDERPINNING WALLS OF BRUNSWICK DOCK.

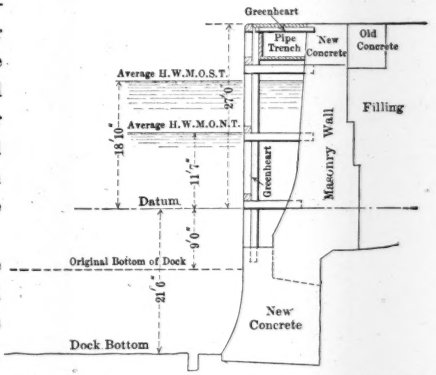
FIG. 4.

From the grip, galleries of an average width of 15 ft., are driven under the wall, on the hit-and-miss principle, so that between each gallery the wall above is supported by a pier of rock about 15 ft. wide. These galleries are filled with 8 to 1 concrete which extends outward as far as the new face line.

After the full length has been treated in this manner, those bays which were missed will be excavated and concreted.

The walls are treated in lengths of between 600 and 700 ft., this length of dam being used over again for further lengths.

Fig. 5 shows the section of the wall after the completion of the underpinning work, and the section of the timber superstructure erected upon piles stepped in the front portion of the new concrete.



SECTION OF WALL AFTER DEEPENING BRUNSWICK DOCK.
FIG. 5.

At a future date the structure will be raised in concrete from its present level of 27 ft. above datum to 30 ft. above datum, a modern shed being substituted for the present one.

The gates and gate clapping-sills are constructed of greenheart, the top of the gates being 25 ft. above datum and the bottom 20 ft. 6 ins. below datum.

The heel-posts, middle heads, and miters, are connected to the cesses by mortises and tenons, and are held together by framing bolts.

Additional strength is obtained by framing plates, on either side and at each end of the voussoirs, secured by dump-bolts.

The voussoirs are connected at the middle heads by 2-in. galvanized bolts, and hoops at the top and the bottom of the gates. There are straps at the back, and connecting timbers at the front, of the gates; the connecting timbers will also serve as fenders to protect the gates from damage when vessels are being passed through the locks. All ironwork, with the exception of straps and framing plates, is galvanized.

Each leaf is supported at the heel-post end by means of a gun-metal cup-and-ball arrangement, working in a socket, and by two bevelled truck wheels, each 1 ft. 2 ins. wide and averaging 3 ft. in diameter, which travel on the iron roller paths laid on the platforms.

At the top of the heel-posts is a wrought-steel collar, 9 ins. deep, and the leaf is held up to the hollow quoin by the collar strap, in which the heel-post is free to move horizontally.

The rise of the granite sill for the 100-ft. gates at the point of miter is 16 ft. 8 ins., and that of the clapping-sill 16 ft. 10 ins. from a line connecting the centers of the hollow quoins.

Each leaf has maximum breadth of 7 ft., and is divided into three voussoirs, the radius of the back of the gate being 92 ft. and that of the clapping face 160 ft. The length along the straight from the center of the heel-post is 55 ft. 3 ins., and the diameter of the heel-post is 2 ft. 4 ins., the breadth of the miter-post at the miter being 1 ft. 8 ins. The depths of the cesses are as follows: 8 ft., 5 ft., 4 ft., 3 ft., 2 ft. 6 ins. and 1 ft. 6 ins., the spaces being 2 ft., 3 ft., 4 ft., 5 ft. 6 ins. and 7 ft., respectively, working from the bottom of the gate. The radii of the gate roller paths are 23 ft. 6 ins. and 47 ft., respectively.

The rise of the granite sill for the 80-ft. gates at the point of the miter is 13 ft. 4 ins., and that of the clapping-sill 13 ft. 6 ins. from a line connecting the centers of the hollow quoins.

Each leaf has a maximum breadth of 6 ft. 6 ins., and is divided into two voussoirs, the radius of the back of the gate being 72 ft. and that of the clapping face 130 ft. The length along the straight from the center of the heel-post is 44 ft. 6 ins., and the diameter of the heel-post is 2 ft. 2 ins., the breadth of the miter-post at the miter being 1 ft. 6 ins. The depths of the cesses are as follows: 6 ft., 4 ft. 6 ins., 3 ft. 6 ins., 2 ft. 6 ins., 1 ft. 6 ins.. 1 ft. 3 ins. and 1 ft. 6 ins., the spaces being 2 ft. 6 ins., 2 ft. 6 ins., 4 ft., 4 ft. 6 ins., 5 ft. 6 ins. and 5 ft. 9 ins., respectively, working from the bottom of the gate. The radii of the gate roller paths are 18 ft. 11 ins. and 37 ft. 9 ins., respectively.

The hydraulic apparatus for actuating the gates consists of horizontal hydraulic machines, placed in pits in the rear of the gate recesses. There are two machines to each leaf of the gates, one for opening and one for closing. The chains are carried down through chain holes, 1 ft. 3 ins. in diameter, to the sheave blocks, from which they are led to staples in the gates.

The dimensions of the machines fully provide for the stresses which may be put upon them in working the gates, with an ample margin of safety.

Machines of four types are in use, *viz.:* Opening and closing machines for the 100-ft. gates; and opening and closing machines for the 80-ft. gates.

Each machine is provided with a main ram, the easy return of which is insured by an auxiliary ram.

The following are particulars of the machines:

The main rams are provided with three pulleys on the ram head and three on the end of the cylinder, the auxiliary rams having two movable pulleys and one fixed pulley.

The machines for opening the 100-ft. gates have rams 1 ft. 7 ins. in diameter and 9 ft. stroke, and the closing machines rams of the same diameter with a stroke of 13 ft. 4 ins. The main chains in each case are $1\frac{1}{2}$-in., and the overhauling chain on the auxiliary ram $\frac{3}{4}$-in.

The machines for opening the 80-ft. gates have rams 1 ft. 5 ins. in diameter and 8 ft. stroke, and the closing machines rams of the same diameter with a stroke of 12 ft. 3 ins. The main chains in each case are $1\frac{3}{16}$-in., and the chains on the auxiliary rams $\frac{3}{4}$-in.

The auxiliary rams and cylinders are fitted to the opening and closing machinery in each case, and have a uniform diameter of 5 ins. and a stroke of 3 ft. 2 ins.

The valves are actuated by levers on the quay level, removable at pleasure, so that there is no projection above the quay level. In addition, there are stop-valves on the pressure and return pipes in the machine pits.

The hydraulic paddles of the main and the branch culverts are actuated by vertical, double-acting cylinders, the cylinders being copper-lined and the piston rods copper-covered.

The dimensions of the cylinders, pipes and valves are such that the machines are capable of lifting the paddles from their closed positions, under the full head of water, through the full lift and lowering them again to their closed positions in one minute.

The paddles are of greenheart, and are connected to the machines by wrought-iron spears dependent on a cast-iron crosshead on the piston rod. In the case of the smaller paddles, 7 x 7 ft. x 10 ins., for the 6 x 6-ft. outlets, the spears are attached to two bolts passing through the paddles from top to bottom. In the large paddles $8\frac{1}{2}$ x $11\frac{1}{2}$ x 1 ft., for the $7\frac{1}{2}$ x 10-ft. inlets, the spears are attached to a cast-iron yoke across the top of the paddle through which the building bolts also pass.

The machines for the branch culverts have 11½-in. cylinders with 6½ ft. stroke, those for the main culverts having 16½-in. cylinders with 10½ ft. stroke.

The valves are worked from the quay level by levers removable at pleasure; there are also two stop-valves in the pits, on the supply and exhaust pipes.

The hand-cloughs are actuated by a screw, suspended from a fixed bearing and worked from a bar head. This screw passes through a brass nut in a cast-iron crosshead, from which a pair of spears is carried down to the paddle.

At the south end of the island, however, the hand-cloughs open into a gallery, in which is a crab winch traveling on rails, so that it can be brought over any clough shaft.

A chain, with bridle, is carried up from each paddle and can be attached to the winch for raising the paddle. The gallery is closed in overhead and thus presents no obstruction on the quay level. In every case, space is allowed in the clough shafts for the withdrawal of the paddles for repairs, etc., without disturbing the machinery.

The superficial areas of the hand and hydraulic paddles are similar; in cross-section, the latter are beveled 1½ ins. to the foot, commencing from a width of 6 ins. at the bottom.

At both ends of each wall, and of the island, 11-ton capstans are provided, six in all, working under a pressure of 800 lbs. per square inch at the starting valves.

The capstans have two powers, 11 tons and 4 tons, respectively, the speed of hauling being not less than 100 ft. per minute under the load of 4 tons.

The engine is connected to the capstan by a clutch, and is arranged so that one lever controls the three operations of putting into action either of the two powers, or disconnecting the engine entirely from the capstan.

The barrel of the capstan is arranged so that hand-power may be readily substituted for hydraulic power, whether or not the engine is connected with the capstan, and the capstan is fitted to receive eight handspikes. Pawls are fitted between the engine and barrel, so that when the capstan is being worked by hand, the men are not endangered.

The hydraulic engines are placed close to the capstan, and are easily accessible for cleaning, packing of glands, and repairs. The starting

valves are worked by levers from the ground level, and are removable at pleasure.

Screw stop-valves are placed on the pressure and exhaust pipes inside the machinery pits, the valves being worked by a detachable key made to suit either valve.

The machinery is designed so that, under ordinary working conditions, no part shall be strained to more than one-quarter of its breaking load.

The dams are designed to give ample strength to sustain at all times the full head of water, the outside faces being constructed and maintained in a condition to allow of vessels bearing against them without risk of injury to either the vessels or the dams.

The temporary timber dams used in connection with the work are of the flitch type, with the exception of those in the river, which are of sheet-piling.

The flitch type of dam has been adopted on the Liverpool Dock Estate as being the best, under certain conditions, for all-round purposes, its great advantage being that it can be used over again without dissection, which reduces the cost of work very considerably when new works and alterations are constantly being carried out. In addition to this, the first cost is not great, and there is generally much less leakage than with other types, especially sheet-piling dams.

Flitch dams consist of vertical timbers connected to walings, spaced to suit the pressure, and supported vertically by means of shores, with a toe of clay on the outside. The walings, generally, are 1 ft. square and 37 ft. long, the horizontal shores being spaced 12 ft. apart between centers.

The flitches are designed for a head of about 30 ft., and usually have seven walings, necessitating seven tiers of shores, the spacing of the latter being as follows: 9 ft., 7½ ft., 5 ft., 3¾ ft., 2½ ft. and 2 ft., respectively, commencing at the top of the uppermost tier and working down, the top of the lowest waling being 1 ft. 6 ins. above the bottom of the flitch.

The skin of the flitch is formed of half timbers about 6 ins. thick, the seams being caulked.

The length of the flitches is such that they can be handled readily, but the shorter the length the greater the number of shores required, as at the joints between flitches double vertical shoring is necessary.

The longest flitch used was 142 ft., and there was no difficulty in placing it; but lengths of about 120 ft. are most suitable, for, beyond this length, the flitches are likely to buckle while being placed, which may cause leakage after the dam has been pumped out.

When the ends are not at walls or the like, enclosures are made by means of gusset flitches, *i. e.*, narrow flitches with walings spaced as before, one end of the gusset being shaped to suit the abutment and the other end vertical. At the vertical end the walings are cut off flush with the skin timber, and at the other end they project about 1 ft. beyond the skin. A double row of sinnet is nailed to either bearing face to make a tight joint when the dam is under pressure.

Put-holes are cut in the abutment to receive the projecting portion of the waling, and, at the other end, the gusset is supported by means of cleats bolted to the walings of the adjoining flitch in such a manner that the skin of the gusset is flush with the end of the flitch.

Paddles are provided, and, as a rule, are fixed in the gussets. As the flitches were launched, they were constructed on their flat, with the walings underneath, the lightest portion of the flitch, *i. e.*, the top, being nearest the water, so as to reduce the strain of launching.

The joints in the walings come in the center of the shores, and care was taken to insure that a number of joints did not come vertically under each other.

Each skin timber is secured to the walings by two $\frac{3}{4}$-in. bolts with their heads countersunk in the walings in order that there may be no difficulty in butting the waling end of the shores. The arrangement of the bolts along the walings is such that each tier has an equal number of bolts in its walings.

In each flitch, for slinging purposes, two whole-baulk timbers are substituted for two half timbers, one at either end of the middle half of the flitch. They are bolted at each waling, and project about 3 ft. above the top of the flitch.

At the ends of the flitch, fillings of the same width and depth as the walings, *i. e.*, 1 ft., are bolted vertically to the skin timbers between the walings in such a manner that the spaces between the walings are made good and a flush jamb formed.

To stiffen the flitch, two horizontal half timbers are run from end to end, one at the top and one at the bottom, in such a manner that their respective bolts pass through the top and bottom walings.

Occasional ring-bolts are placed along the top and bottom walings, so that rings occur on either side of the flitch.

Pig or scrap iron, or old rails, used for ballast, are placed between the two lower walings, and are secured by means of vertical deals spiked to them.

While the flitches were under construction, the work of preparing and cleansing the site for the dam was in progress. A grab dredger removed as much silt as possible, afterward a bucket dredger worked over the site to pick up anything hard and to level a bed on which the flitches were to rest; a scraping flat followed, and afterward a suction flat.

The flitches were placed by means of a floating crane, and the gusset flitches by derrick flats.

Put-holes usually formed the support for the shores at the land end, the other ends being supported by cleats bolted to the shores, the overhang of the cleats being sufficiently long for bolting to the walings, this precaution being taken to prevent the shores from floating out in case of a "blow."

The flitches are secured at the top by pennants made fast to the ring-bolts, and at the bottom by means of toggles made from old rails or bar iron bent into the form of a letter J, the long leg of the toggle being placed in a hole drilled in the rock about 6 ins. from the dam.

The bottom walings are first shored opposite the toggles, after which packings are fixed between the toggles and the flitch. The shoring is then completed, all shores placed by divers being weighted by means of pig iron or old rails spiked to either side of the shores.

Any very hollow places under the flitches were made up by means of concrete, and, when the space was small, bags of clay were placed in the angle formed by the flitch and the bottom.

Concrete blocks are placed about 18 ft. away from the face of the dam and along its entire length, in such a manner that a revetment is formed for the clay.

The blocks, which are 6 x 3 ft. and 3 ft. deep, are laid lengthwise with the dam, and the clay is placed between the dam and the concrete blocks to a height of about 8 ft. at the flitch, tailing off to the depth of the blocks.

In the case of long shores, lacings are provided, also "toms" to give vertical support to the shores.

The butts of the shores are in all cases cut square, and, in the case of diagonal shores, cleats, bevelled to suit the angles formed by the shores and the walings, are spiked to the latter.

The position of the pumps is such that the work below dock water level may be completed without their removal. Whenever possible, the pump framing is quite independent of the dams, so that, in case of accident to the latter, the pumps will not be disturbed.

After the divers had finished their shoring and the clay had been well rammed, before an attempt was made to pump out, a careful examination of all the under-water shores was made to ensure that they were all in place and that none was under too great a strain.

As regards maintenance, with a view to minimizing the dangers of "blows," soundings are taken from time to time at regular intervals along the entire length of the dam, to guard against any fretting of the clay.

DISCUSSION.

ALFRED NOBLE, President, Am. Soc. C. E.—This very interesting paper refers to a class of work which supplies a large field for engineering practice on the other side of the Atlantic. The magnificent dock system at Liverpool, developed to meet the severe tidal conditions there, has no parallel on our Atlantic coast, where the tidal range is much less and enclosed docks do not furnish the most economical solution of the problem of handling a large commerce. These different conditions render the paper all the more interesting, but, at the same time, make its general discussion difficult for American engineers. Mr. Noble.

The paper, however, describes many features of construction of great interest wherever subaqueous work is carried on, of which the speaker desires to note briefly two or three. The ingenious and successful method of laying dry the face and base of a wall to be underpinned, is worthy of most careful study.

The speaker had an opportunity, a few years ago, to see some of the work described, or some of similar character, in the vicinity, while in progress. The concrete was much wetter than is usual in this country, and might be described as sloppy. The materials were first mixed dry, and then, with the requisite amount of water, passed through a very simple gravity mixer. A large graving dock, built of similar concrete and finished a year or two before, was visited and found in excellent condition. The chamber was about 900 ft. long, and the entire masonry more than 1 000 ft. long. In the entire length, only one small crack attributable to contraction could be found. In view of experience in this country, respecting the expansion and contraction of concrete with changes of temperature, this result seems remarkable. Until within a few years, specifications for concrete in this country usually required a mixture so dry that moisture would show on the surface only after hard ramming. English engineers freed themselves from this pernicious practice earlier than did American engineers.

The use of wood for large lock gates seemed to the speaker remarkable when first informed of it, but it was stated that greenheart timber was teredo-proof and was considered even more durable than steel. At Liverpool the presence of sea water polluted by sewage renders the conditions particularly unfavorable for iron or steel, while the great durability of greenheart, and the greater facility with which timber gates can be repaired in case of accident, have apparently turned the scale in favor of wood throughout the entire system of docks at Liverpool. This practice has been followed at the Manchester Canal, but is not by any means general throughout Great Britain, while, on the

Mr. Noble. continent of Europe, steel is generally adopted. A somewhat similar statement as to the relative durability of steel and wood was made to the speaker with respect to railroad ties on the Panama Railroad. These were of one of the many varieties of lignum vitæ. The timber is so hard that holes must be bored for spikes. The speaker was assured that many had been in service thirty to forty years, and were removed, not for decay, but because the rails had cut deeply into them. This durability is particularly noteworthy, in view of the rapid decay of ordinary timber in that hot, moist climate.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—Few harbor engineers in this country have had any experience in the line of work laid out for the proposed improvements at the Brunswick Docks, Liverpool. Nevertheless, this paper is extremely interesting, and is full of valuable instruction.

These vast harbor improvements were, no doubt, rendered necessary by the growing traffic in the Manchester Canal, which, at last accounts, was certainly detracting considerably from the natural growth of Liverpool Harbor. The magnificent masonry dock walls surrounding the different basins will certainly excite the admiration of all engineers. The writer thinks, however, that the American harbor engineer would do differently under somewhat similar circumstances. Instinctively, he avoids conflicts with uncertain foundations, and, instead of building a massive concrete dock-wall, would proceed to make the basin excavation, with a hydraulic dredge, to the required depth, allowing the natural slopes to take their own shape. The material excavated would be pumped into those portions of the work which needed filling. He would then build a pile structure from the top of the slope out to the foot, and then proceed to lay thereon the necessary railroad tracks and build proper freight sheds, etc., exactly as would be done in the case of a solid concrete masonry quay wall. Such a simple structure would be vastly cheaper, and, at the same time, answer all the practical requirements perfectly.

The query might naturally be made, would not the annual repairs on such a temporary structure be so great in amount as to impair its usefulness? The writer thinks they would not; inasmuch as the foundations of creosoted piles would not ordinarily need attention for fifteen to twenty years, and the chief item of expense would be in keeping up the repairs of the natural wear and tear of the wooden flooring of the wharf. This could be reduced materially by laying a wooden block pavement thereon, which would answer all the requirements of heavy traffic. When the time arrived for making repairs underneath the sheds, this could be done easily by means of a special pile-driver, built within the shed itself, by which the new pile could be hoisted up through holes made in the roof. By this means, all repairs to pile-work could be made cheaply, and without much inconvenience to traffic.

Mr. Le Conte.

The author refers to faulty borings indicating 30 ft. to bed-rock, whereas it was actually found at 68 ft. The writer has had much the same vexatious experience, and he takes this occasion to state that, in all cases where the surface of the so-called bed-rock is at all irregular, there is nothing as misleading as a system of detached test-borings, so much so, that, in contract work, the engineer in charge too often wishes he had never made any preliminary borings. After all, they show absolutely nothing outside of the particular point at which each one is made, and, consequently, furnish really very limited information.

The writer is interested greatly in the many important details of construction described so clearly by the author. The practice has now become quite common to mix boulders with the mass of concrete, and thus save cement mortar to a very material extent. Where the concrete is laid in thick heavy masses the boulders can be placed not more than 20 ins. from each other and about the same distance from the forms, so as to insure good ramming between; thus the saving in cement becomes very great indeed, and, at the same time, does not detract in the least from the excellence of the concrete.

The author's statements, as to testing the cement before it was accepted, are very instructive and seem to cover the ground most thoroughly.

One feature, however, impressed the writer, *i. e.*, as to the richness of the concrete used in the dock walls. The poorest grade used was 8 to 1, which the writer presumes means 1 to 2 to 6; *i. e.*, 1 to 2, mortar and 6 of the aggregates. This would seem to be abnormally rich for the plain massive dock walls, the main feature desirable being dead weight, the unit strength of the material being a secondary consideration; hence it is apparently uncalled for. Of course, in a large piece of work like this, a comparatively trifling variation in the proportions of the concrete would easily make a vast difference in the cost of the cement. That is where good sound judgment comes into play.

Mr. Low.

Emile Low, M. Am. Soc. C. E. (by letter).—The writer is interested particularly in the description of the concrete work. He notes, on page 48, the composition of the various grades of concrete, consisting of approved aggregates of gravel, broken bricks, or stones, with a proper proportion of sand, mixed with Portland cement of the best quality, in the various proportions of 8 to 1, 6 to 1, and 4 to 1, the larger figures denoting the parts of the aggregates; the smaller ones, the volume of cement. He infers from this that most of the concrete was made from a natural mixture of gravel and sand to which the cement was added.

During 1892 to 1896, the writer was the Engineer of the Mathieson Alkali Works, of Saltville, Virginia. Nearly all the construction in

Mr. Low. these works was done by English artisans, who came from the neighborhood of Liverpool, and some had been engaged in the building of the Manchester Ship Canal.

At Saltville considerable concrete was used, especially in foundations, and it was there that the writer received his first lessons in wet concrete.

Owing to the preponderance of the English or foreign element in the management, English ideas as to methods of execution naturally prevailed, and the work was done as on the "other side." With hardly an exception, the concrete was made from a natural mixture of gravel and sand, obtained from the bed of the nearby Holston River, and hauled to the building sites in wagons, the term wagon here being used in the accepted American sense, a wagon load being somewhat less than a cubic yard, the exact amount not being recalled.

This aggregate was dumped upon a plank platform, leveled off to a uniform depth of about 6 ins., on top of which the requisite quantity of American Portland cement (Atlas) was spread. The materials were then turned over several times with shovels, after which water in generous quantity was added, the mass being again turned and then shoveled to the place of deposit.

The writer recalls the building of a concrete foundation for a brine reservoir, the depth of the concrete in the trench being 1 ft., the concrete being made so wet that the man in charge of the leveling actually waded about in it, his feet being encased in rubber boots. The concrete took about the usual time to set. On the completion of the reservoir walls, which were of brick, the floor or bottom was put in. This was joined to the foundation just mentioned. Before joining the floor it was thought best to roughen the old concrete, especially sharpened tools, picks, bars, etc., being used, but the points of these turned as if made of lead, no impression whatever being made on the concrete. This roughening was then abandoned and the surface of the concrete merely cleaned with water and brooms. All other concrete was made wet, and the writer cannot recall a defective piece of work due to an abundance of water being used.

It is not to be inferred from this description that the writer countenances the extreme use of water in making concrete, but he desires to say that his predilections are for what is called a wet mixture, the experience gained from his English brethren having been of a convincing nature.

Mr. Noble, in his discussion, describes the concrete used at Liverpool as sloppy. The writer thinks that this definition applies admirably to that used at Saltville, and is glad to know that Mr. Noble is an advocate of wet concrete, and that he calls dry concrete by no stronger term than pernicious practice.

As the presentation of methods concerning concrete mixing are always of an interesting nature, a few facts concerning the concrete

bottom of the Hiland Avenue Reservoir of the Pittsburg Water-Works, with which the writer had some connection, will be briefly given. Mr. Low.

The specifications prescribed that the concrete should be composed of 1 part of hydraulic cement mortar and 2½ parts of clean gravel or broken stone, free from dirt, and broken so as to pass through a 2½-in. ring.

The mortar was to be prepared from American hydraulic cement, of the best quality, and clean, sharp sand, free from loam. It was to be mixed dry, in the proportion of 1 part of cement to 2 parts of sand, and a sufficient quantity of water added to produce a paste of proper consistency.

As previously stated, the concrete was used for covering the bottom of the reservoir, the same being 5 ins. thick. The mode of making the concrete was as follows: Suitable mortar boxes were used. These were about 4 x 6 ft. in size, and supplied with legs to keep them about 2 ft. above the ground. In these boxes the cement and sand were mixed dry, and then water was added to make a creamy paste. Close to the box a plank platform was laid on the prepared puddle foundation. On this the broken stone was spread, after which the mortar was allowed to run out over the stone, the thicker portions being shoveled out. The mass was then turned over thoroughly and mixed, after which it was deposited. No forms were used. Small stakes, the tops conforming to the proper elevation, were set at frequent intervals, and the concrete was deposited so as to correspond with these, the true height being obtained mainly by thorough ramming. The small stakes were withdrawn as the work progressed. The quantity of concrete thus placed was more than 11 000 cu. yds., corresponding to an area of about 16 acres.

The writer is interested very much in the author's description of the temporary timber dams used in keeping the dock work dry. Figs. 4 and 5 show these so-called flitch dams, generally, but it is to be hoped that the author will give some detailed drawings, so that their construction can be understood better. Some notes or facts concerning the leakage of these dams would be of general interest.

George Cecil Kenyon, Assoc. M. Am. Soc. C. E. (by letter).—With regard to Mr. Noble's remarks comparing the relative endurance of greenheart and steel gates, there can be no question that the experience in Liverpool is entirely in favor of the timber as against the metal. Mr. Kenyon.

As regards first cost, there is very little to choose between them, but with regard to maintenance and endurance, within the limits of the experience at Liverpool, greenheart timber leaves nothing to be desired. As an instance of this, it may be stated that in the reconstruction of certain dock works, recently, in another part of the estate, it was found necessary to take down some greenheart gates and erect new ones of a larger and heavier type. The gates were constructed

Mr. Kenyon. with iron bolts and stays, and were erected originally in the year 1856. When the gates were taken to pieces, the timber was found to be as sound as on the day it was put in, and, as far as was possible, having regard to the scantlings required and available, was entirely reused in the construction of the new gates. The iron bolts and stays, on the contrary, showed serious decay, and it was not possible to bring them into use again for the purpose of the new gates.

With regard to Mr. Le Conte's remarks, his assumption that the vast harbor improvements were in consequence of the growing traffic of the Manchester Ship Canal is not warranted by the facts of the case, and it hardly seems a logical deduction to suggest that when the traffic of the Port of Liverpool is diminishing, it becomes necessary to spend large sums in fresh improvements. As a matter of fact, the progress of the port of Liverpool has been uninterrupted for very many years, and for the last thirty years there has been a constantly increasing capital outlay in the endeavor to overtake and meet the increase of trade. This outlay is also in an increasing ratio, and, in spite of that, the outcry for increasing accommodation is as loud to-day as it has ever been, in the history of the port. As an instance of this, it may be stated that from July 1st to December 31st, 1903, the increase in the tonnage of ships frequenting the port has been 800 000, an amount sufficient in itself to constitute the trade of a good-sized port.

Mr. Le Conte suggests a piled structure, to form a quay and take the place of the walls adopted, and claims that that would get over the difficulty of the deep foundations described in the paper. In making this suggestion, however, he misses entirely the point that the foundations in question were not encountered in the case of the construction of quay walls, but in the walls of the entrances to locks; the construction which he suggests, of course, is out of the question in the latter case. The dock walls in this vicinity are generally of extremely light section, owing to the fact that the rock is encountered at a somewhat high level, and the lower part of the wall is merely a facing to it.

The construction of piled wooden quays, has been carried out in England, in several instances, but is quite unsuitable for such heavy traffic and loads as those dealt with by the quays of the Liverpool Docks. This can be understood readily when it is stated that the ordinary quay working load is 3 tons per square yard, and that concentrated weights from the double-story sheds, amounting to 280 tons, have to be provided for, and, in the case of treble-story sheds, 540 tons from one column.

Timber blocks, as a pavement, would not be at all suitable for the heavy cart traffic frequenting the quays of the docks. As a matter of fact, the only pavement which stands the traffic, without undue cost in maintenance, is granite paving setts on a concrete foundation.

Mr. Kenyon.

In one or two cases, where the conditions were suitable, quays have been already constructed at Liverpool on groups of greenheart piles finished off with ferro-concrete and paved with granite setts. Where the surrounding ground is formed of fairly suitable material, and a heavy section of dock wall, if constructed, would be necessitated, no doubt this form of structure could be adopted with economy and advantage.

Dredging out excavations of the body of the dock is also resorted to, in cases where the circumstances favor such a course, but cannot be used generally.

Mr. Le Conte's assumption as regards the richness of the concrete mixture forming the dock walls is also based on a misapprehension of the conditions. The original mixture on the platform, where hand mixing is used, or in the hoppers, where machines are used, is 8 parts of gravel, formed of coarse sand and pebbles of varying sizes, to 1 part of cement. After this is put into the wall moulds, rough lumps of stone, varying from sizes which can be handled by a man, to large stones several tons in weight (from old masonry, when such material is available), are bedded into the concrete, and form probably in the aggregate about 30% of the total volume of the finished work. This, so far from being "abnormally rich," forms a very sound, massive structure which can be executed at a cheap rate.

The following extract from the Annual Statement of the Chairman of the Mersey Docks and Harbour Board, at the closing meeting of the year (1903), will bring the position of the work treated in the paper up to date:

"At the Brunswick Dock the east wall had been underpinned and the raising of the coping had been completed. They had bought from the railway companies their portion of the Brunswick Branch Dock and the land adjoining it and the dock itself. That branch dock was being filled up. A new shed, 1283 ft. long and 150 ft. wide, with a roadway and railway lines, was being constructed on the east side of the Brunswick Dock. The bottom of the dock was being deepened by dredging to an additional depth of 12 ft., which would give a level of 21 ft. below the Old Dock Sill. The new river entrances at this dock were practically completed; one of them was 100 ft. wide and the other 80 ft. wide, with sills 19 ft. 6 ins. below Old Dock Sill. The water would soon be admitted. The fairway in the river to these entrances was being dredged by the new dredger *Coronation*, which had a capacity of 3 500 tons. Those who had not been down to see these new entrances should, he thought, do so, for they were a most magnificent piece of work. It might be said that in some respects they were a duplication of those at the Sandon Dock, but he had been told by experts that these entrances presented greater facilities for docking ships than any entrances into any docks elsewhere; and what was also very interesting was the fact that these two entrances would provide—he meant the Sandon and the Brunswick entrances—for the entrance with the greatest ease of the largest steamers at present contemplated.

Mr. Kenyon. "They had heard quite lately—it might not be true, but he had seen it in print—that it was contemplated to build new steamers for the American trade, 800 ft. long and of proportionate depth and breadth, and these entrances would allow such steamers to enter with the greatest facility, and there would be ample room for them after they had got in."

In reply to Mr. Emile Low, the writer desires to say that there should now be no question as to what is meant by the various grades of concrete used at Liverpool, this point being stated clearly in the paper, and again in the writer's reply to Mr. Le Conte.

In the bulk of the concrete put in under the writer's supervision, the men spreading it in its ultimate position are invariably ankle deep in the concrete, and wear water-tight leather knee-boots in consequence. By having a sloppy mixture, the concrete can be well worked against the lightly constructed moulds, without displacing them, and a good, glossy face obtained on the exposed surfaces. Another reason for using sloppy concrete is that more "plums" can be placed in the mass, the sloppy mixture allowing the plums to settle down into it and also bonding much more readily than in a drier mixture. Even with the sloppy mixture, the moulds were in many cases stripped within 48 hours after mixing.

In addition to washing, the writer is also in favor of roughening older concrete surfaces before joining them up with newer concrete, as roughening after washing ensures that all grease used for paying the moulds is entirely removed. The writer is of the opinion that perfect joints in concrete are a fallacy, and his experience teaches that the different beds can always be traced, no matter what precautions have been taken, even if the interval between the successive layers has only been a meal time.

A paper by the writer, entitled "Temporary Dams,"* is in the Library of the American Society of Civil Engineers, and gives detailed information in relation to "Flitchy Dams," as used on the Mersey Dock Estate. Particulars regarding leakage from these dams can also be obtained from the same source.

In conclusion, the writer desires to express his pleasure that, apparently, his paper has been so well received, and is amply repaid for the time spent in its preparation, if it has benefited, ever so little, any of the members of the Society.

* Excerpt from the *Transactions* of the Liverpool Engineering Society, Vol. XXIV

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 971.

THE BREAKWATER AT BUFFALO, NEW YORK.*

By Emile Low, M. Am. Soc. C. E.

With Discussion by Messrs. Thomas D. Pitts, William T. Lyle, D. E. Hughes, G. H. Raymond, George E. Fell and Emile Low.

The City of Buffalo, N. Y., situated at the foot of Lake Erie, is practically the eastern terminus of most of the lake traffic, the natural barrier intervened by the Falls of Niagara River preventing water communication with Lake Ontario, except through the inadequate Welland Canal. This natural obstruction has aided in building up a large commercial city, with an immense water traffic.

The present harbor of Buffalo may be said to be made up of two parts, one comprising the inner harbor, formed by the natural stream, Buffalo Creek, and the artificial channel, named the City Ship Canal; and the other, the outer harbor, formed by a chain of breakwaters, extending southeast along the lake front, from Fort Porter to Stony Point, a distance of more than 5 miles.

With hardly an exception, the grain elevators, freight and storage warehouses, and coal-transfer trestles, are on the banks of the first-named, inside waterways, affording ready water communication, while access to railroads is provided by tracks, paralleling the waterways in the rear of the above-named structures.

The practical completion of the stone (or rubble-mound) section of the breakwater, in December, 1902, forged the last link in the chain

* Presented at the meeting of December 16th, 1903.

of a system of harbor protection which is undoubtedly without parallel in the United States. This structure is a part of the Buffalo breakwater, known as the Extension to Stony Point, and has been building since 1897, under the direction of Major (now Colonel) Thomas W. Symons, M. Am. Soc. C. E., Corps of Engineers, U. S. Army. J. C. Quintus, M. Am. Soc. C. E., Principal Assistant United States Engineer, of the Buffalo District, was associated with Colonel Symons and his predecessors in the design and execution of the Buffalo breakwater work back to 1894. It may be opportune to state here that much of the matter contained in this paper has been gleaned from Government reports by Colonel Symons and his predecessors, especially by Captain F. A. Mahan (now Major, retired), as well as from information and data collected by the writer in his official capacity as United States Engineer in immediate charge of the Stony Point extension.

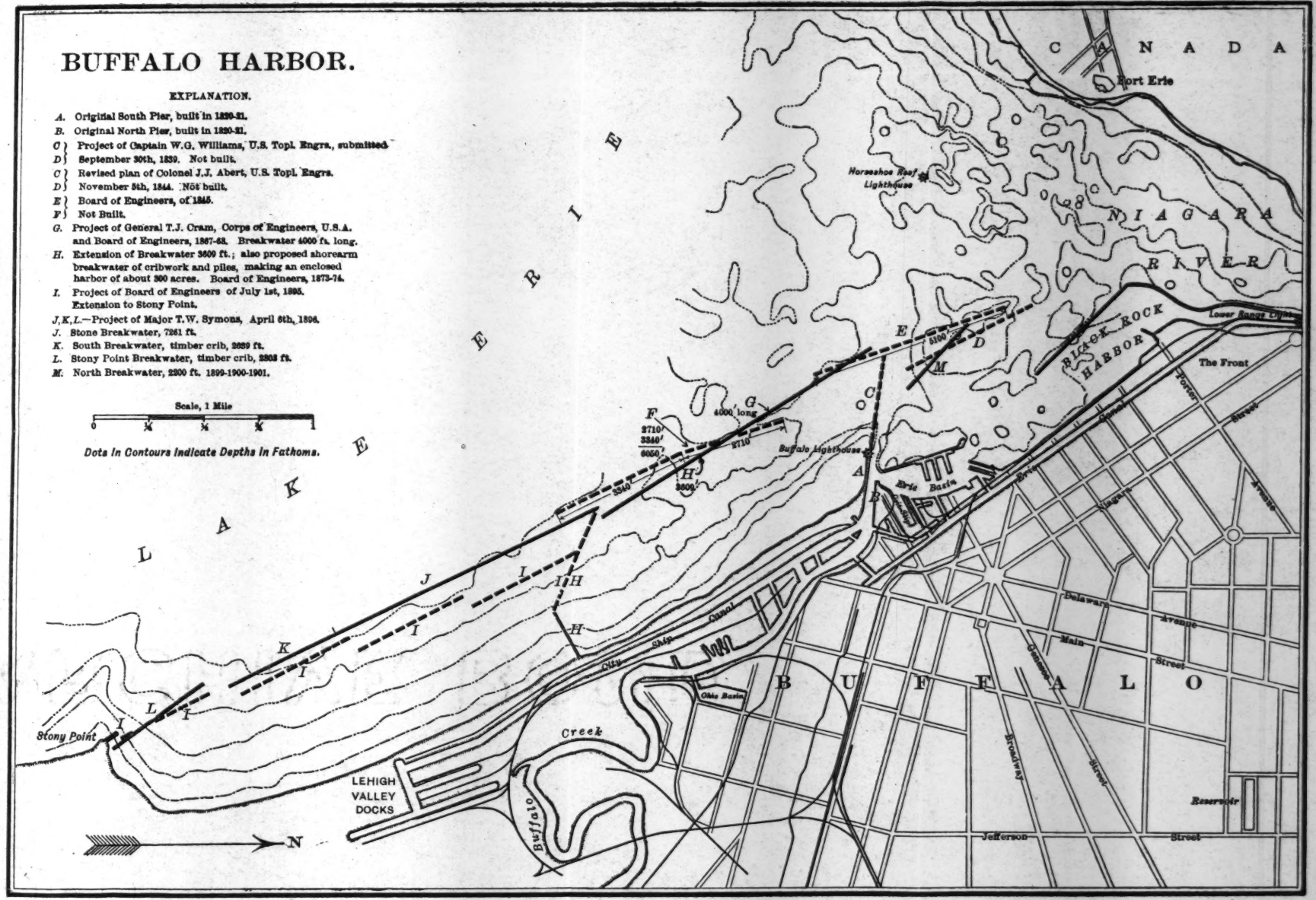
Historical.

As Buffalo faces the open lake, it possesses no natural harbor. The only safe place, originally, where vessels could lie, or could receive and discharge cargoes, was the area within the mouth of Buffalo Creek, which stream, previous to the construction of the harbor, was exceedingly difficult to enter, a long outer bar of gravel confining its channel inshore, and causing its entrance into the lake to be nearly parallel with the shore. For this reason the mouth of Buffalo Creek was closed to navigation most of the year, even to the smallest vessels of the time. Freshets, occasionally, opened channels across the bar, but these were soon closed again by the waves of the lake. Thus, the current of the stream and the action of the lake caused a continual shifting of the bar.

The earliest record as to the navigability of this harbor dates back to April 10th, 1818, when the Legislature of the State of New York passed an act, authorizing the survey of Buffalo Creek, with a view of building a harbor, and directing that the surveyor should be paid $3 a day for his services.

The survey was made gratuitously by the Hon. William Peacock during the summer of 1818. In order to build the harbor, the State, in 1819, loaned $12 000, on proper security, to a committee of citizens. Samuel Wilkeson superintended the construction, which was commenced in 1820, and finished in 221 days in 1821. See piers marked *A* and *B* on Plate V.

PLATE V.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LII, No. 971.
LOW ON BUFFALO BREAKWATER.



The work consisted of two parallel piers at the mouth of Buffalo Creek. The south pier was of hewn timber cribs filled with stone, and extended one-quarter mile into the lake, to 13 ft. of water. The north pier was a double row of piles filled with brush and sand. The piers concentrated the current, and practically removed the bar. The original cost of this work was $14 000.

Here it may be proper to say a few words regarding the *Walk-in-the-Water*, the pioneer steamboat above Niagara Falls. This boat was built at Black Rock, and launched on Thursday, May 28th, 1818. It was about 150 ft. in length and of 28 ft. beam. The motive power was a square, or cross-head, low-pressure engine, quite complicated in its parts, with ponderous fly-wheels and a superabundance of counter-balance weights. The main cylinder was 40 ins. in diameter, with 4-ft. stroke. The boiler was 14 ft. in diameter, with two large direct flues, and four return flues. The paddle-wheels were 15 ft. in diameter, with buckets 5 ft. long. The average speed of this boat, in favorable weather, was about 6 or 7 miles per hour. It was also rigged with two masts for carrying sail. The boat made several trips to Detroit, Mackinaw and Green Bay, and was wrecked on Thursday, November 1st, 1821, near the present south pier, during a violent gale.

In 1825 the Erie Canal was completed to Buffalo, and in 1826 the piers at the mouth of Buffalo Creek passed into the control of the United States, who rebuilt and have ever since retained charge of them.

The various projects for the formation of a harbor at Buffalo are summarized as follows:

For a proper understanding of this summary, reference is made to Plate V, a map of the harbor and a portion of the city fronting thereon.

In 1830 it was determined to construct a cross- or sea- wall nearly at right angles to the eastern end of the south pier, to prevent the sea from making a breach there. In 1835 Lieutenant T. S. Brown, U. S. Engineers, submitted a project for the formation of a spacious harbor, by extending the Black Rock breakwater from Bird Island to within 1 000 ft. of the new Buffalo Lighthouse. This proposed harbor would have occupied that portion of the lake front now bounded on the north by Porter Avenue and on the south by Genesee Street. This

project was not adopted at the time, but was partially realized later by the construction of the Erie Basin and the Bird Island Pier by the State of New York. The latter structure forms part of the Erie Canal system, its purpose being to divert some of the waters of Lake Erie into what is called Black Rock Harbor, which, virtually, is the main feeder of the Erie Canal.

At the present time the United States Government is improving that part of the lake front lying between the south end of Bird Island Pier and the Erie Basin, by dredging it to 23 ft. This project is known as the Improvement of Lake Erie Entrance to Black Rock Harbor and Erie Basin, and was authorized by the River and Harbor Act of June 13th, 1902.

In 1837 it was proposed to extend the sea-wall southward one mile. This work was duly executed. Its location is along the beach, in a southeasterly direction from near the mouth of Buffalo Creek. Its object was to prevent the encroachment of Lake Erie. A portion of the work is an earth mound faced with masonry, the remainder is a masonry structure. Its construction was stopped in 1867, at which time it had a total length of 5 720 ft., of which there were 4 080 ft. built with coping, 1 319 ft. without coping, and 321 ft. of foundation only.

On September 30th, 1839, Captain W. G. Williams, Corps of Topographical Engineers, submitted a project for the formation of a harbor north of the mouth of Buffalo Creek, to be consummated by extending the south pier into 23 ft. of water and building a detached breakwater to protect the entrance. See projects marked *C* and *D* on Plate V.

On November 15th, 1844, Colonel J. J. Abert, Corps of Topographical Engineers, reviewed this plan, stating that there is, in fact, no harbor, properly speaking, at Buffalo. There is an extensive line of wharf and dock room in the creek, at which vessels load and unload, but there is no harbor within the protection of which a revenue or national vessel can lie at anchor.

The plan recommended the construction of a breakwater upon the so-called Horse Shoe Reef, about 2 650 ft. from the northern shore; the breakwater to be about 3 700 ft. long; its western end to be about 2 100 ft. from the Black Rock pier; and its eastern end about 800 ft. from a proposed extension of the south pier, this to be extended

about 2 400 ft., so as to throw its southern extremity sufficiently beyond the breakwater on the Horse Shoe Reef. This would have given a sheltered harbor, 7 400 ft. long and 2 100 ft. wide, throughout the greater part of which there would have been from 14 to 20 ft. of water.

In 1845 a Board of Engineers considered two projects for an outer harbor. The first, for a detached breakwater, 5 100 ft. long, north of the lighthouse pier, the second, also for a detached breakwater, but south of the same locality, 6 050 ft. long. See project marked *E-F* on Plate V. The Board recommended the south breakwater as the most suitable, and also reported that should the demands of commerce require its extension at some future period, it might be extended even to Four-Mile Point, *i. e.*, Stony Point; and, also, that the northern one might some day become a necessity. See Fig. 1.

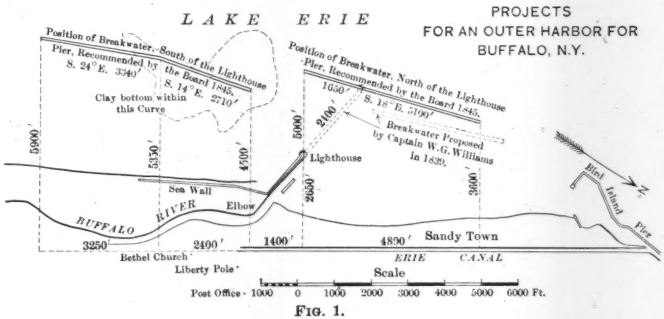
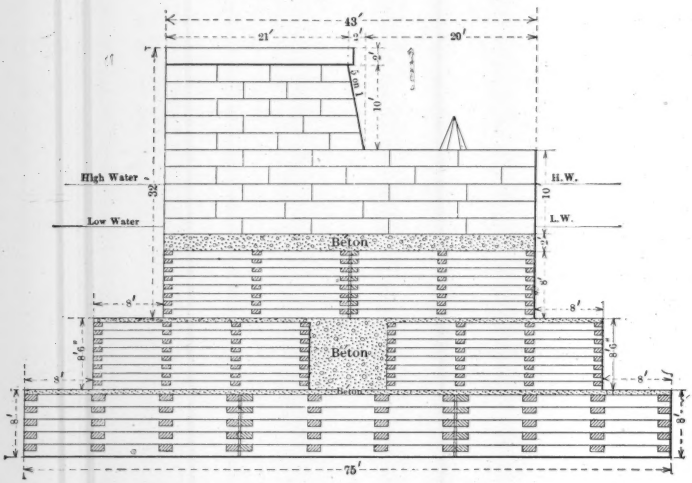


FIG. 1.

It is a matter of interest to note that the lines of these proposed breakwaters approximate closely the location of the Old and North Breakwaters, as actually constructed a number of years afterward, and the prophecy of the extension to Stony Point has already become an accomplished fact.

The type of structure recommended by the Board of Engineers of 1845 is shown by Fig. 2. Its essential features were three cribs under water, surmounted by a masonry structure. The lower tier (resting upon the lake bottom) to be 75 ft. wide, and composed of five courses of 18-in. square timbers. The next tier to be 59 ft. wide, eight courses high, of 12-in. square timbers. The top tier to be 43 ft. wide, having eight courses of 12-in. square timbers. The cross-section of the

masonry, as shown by Fig. 2, was to be built on top of the last tier, with its base below low-water level. The face stones were to be 10 ft.



CROSS-SECTION
PROPOSED TYPE OF BREAKWATER
FOR AN OUTER HARBOR
FOR BUFFALO, N.Y.
FIG. 2.

long, 5 ft. wide and 2 ft. thick, laid alternately as headers and stretchers. The cost of 100 lin. ft. of this proposed structure was estimated as follows:

First tier of cribs	$3 440.60
Second " " "	2 984.60
Third " " "	2 536.94
Masonry	13 265.00
	$22 227.14
Contingencies, 10%	2 222.70
Total cost for 100 lin. ft	$24 449.84

This project was never carried out.

Meanwhile, the City dredged Buffalo Creek for an interior harbor, and, in 1848, extended its area by digging the Blackwell Ship Canal, 200 ft. wide and 16 ft. deep.

In June, 1867, General T. J. Cram submitted a project for the enlargement and repair of the harbor. This was modified in December, 1867, and in 1868 the modification was referred to a Board of Engineers, which recommended certain repairs to the piers at the mouth of Buffalo Creek and the deepening of the channel between them, which work was duly executed. This board also proposed the shortening of the Erie Basin breakwater, and the construction of a canal from Buffalo Creek to the lake, about 1½ miles south of the south pier. These two items were rejected. However, the most important recommendation of this board was the project for a detached breakwater, in about 27 ft. of water, beginning at a point in the prolongation of the south pier and 2 500 ft. from the lighthouse, thence running southward on a line parallel with the shore for 4 000 ft. See project marked *G* on Plate V. The structure was to be of cribwork filled with stone and decked with plank; to have a rectangular cross-section; the width in 29½ ft. of water to be 34 ft., and in no case to be less than the total height from the bed of the lake to the top of the deck; and the top to be 5 ft. above the highest water known.

In accordance with this recommendation, the construction was duly authorized, and the contract for the northern end let to Messrs. Bailey and Denny. The first crib was sunk on June 7th, 1869, from which date begins the creation of Buffalo's outer harbor.

A Board of Engineers, convened in 1873 and 1874, recommended the construction of a sand-catch pier, to be formed of a pile pier, 10 ft. wide, covered by 6 ft. of superstructure, and extending to the 12-ft. contour in the lake, thence prolonged in the same direction to a total length of 1 270 ft. by cribwork, 20 ft. wide, having a superstructure 6 ft. high, the shore end of the pile pier to be connected with the railroad bulkhead behind it, by a close row of piles. This pier was to begin at a point opposite the south end of the City Ship Canal.

The Board also recommended that the detached breakwater be lengthened 3 600 ft., thus giving it a total length of 7 600 ft., instead of 4 000 ft., as adopted by the project of 1867. A shore arm was added,

built so as to leave a fair-weather opening of 150 ft. between it and the main structure, and to have a direction of 45° with the shore line until it reached the sand-catch pier, prolonged to meet it. See project marked *H*, on Plate V.

The sand-catch pier was to serve the double purpose of arresting the movement of sand along the shore (a movement tending to shoal the entrance of Buffalo Creek), and also to close the south end of the new harbor, thus making, with the completed breakwater, a basin with a smooth anchorage ground of 300 acres outside the 16-ft. contour. The sand-catch pier was begun in the fall of 1874, and was completed in 1897, having reached a length of 1 148 ft.

The shore-arm breakwater, authorized in 1874, and, in connection with the sand-catch pier, intended to form the southern boundary of the proposed harbor, was begun in 1893. Of the 1 150 ft. under construction, 800 ft. had practically been completed, when, on October 14th and 15th, a severe storm from the southwest wrecked the entire structure. For more than 24 hours during this storm the wind velocity was greater than 40 miles per hour, reaching a maximum of 60 miles.

Up to this date, the various approved projects for a harbor, with their modifications, may be summarized as follows: The two piers at the mouth of Buffalo Creek, the north pier, 775 ft. long, and the south pier 1 475 ft.; a masonry sea-wall 5 720 ft. long, running southeast from the south pier, a detached breakwater, 7 600 ft. long, parallel to and about one-half mile from the shore; a sand-catch pier, 1 148 ft. long, and a shore arm, of a proposed length of about 3 000 ft., to close the south end of the space between the breakwater and the shore.

The wrecking of the shore arm directed attention toward an increase in the harbor area. In 1894 the extension of the breakwater to Stony Point received considerable public attention. On July 1st, 1895, a Board of Engineers submitted a plan for this extension, the essential features of which were: The construction of five isolated breakwaters, three to be each 2 900 ft. long, one of 2 400 ft., and a shore arm of 400 ft., a total of 11 500 ft. They were to be separated by openings about 350 ft. wide. The structure recommended was a rubble mound resting upon an artificial foundation to be secured by a trench excavated to bed-rock, and refilled with gravel to the lake bottom. On this foundation it was proposed to build a structure of small rip-

rap to a height of about 12 ft. below mean lake level, and upon this to build with large stone, weighing from 4 to 10 tons, up to about 10 ft. above that level. The large stones were to be deposited irregularly in a wall along the axial line of the proposed structure and with such side slopes as could conveniently be made to stand. It was expected that the action of the waves would reduce the height and flatten the slopes down to a position of ultimate stability. The cost of the proposed improvement was estimated at $2 200 000. See project marked *I* on Plate V. See also Fig. 11, page 113.

The project of the Board of Engineers was reviewed critically by Major T. W. Symons, who assumed charge of the Buffalo District on October 14th, 1895, and who modified the plans materially. The changes made were the building of two breakwaters, in place of five, with only one opening of 600 ft. between them. See project marked *J*, *K*, *L*, on Plate V.

The breakwater was also located farther seaward, thus adding a considerable area to the harbor. The main breakwater was to have a length of 9 700 ft.; 5 000 ft. to consist of the rubble-mound type, and the remaining 4 700 ft. to be timber cribwork filled with stone, on an artificial foundation, obtained by excavating a trench and refilling it with sand and gravel. The other breakwater was to jut out from the shore at Stony Point 2 800 ft., and consist of timber cribwork founded in a manner similar to the main section. See Figs. 12 and 13, page 113.

The work was duly authorized by the River and Harbor Act of June 3d, 1896. The contract was let to Messrs. Hughes Brothers and Bangs, of Syracuse, N. Y., on January 27th and 28th, 1897, for $1 765 450.63.

The most northerly government breakwater at Buffalo is called the North Breakwater, and is a structure 2 200 ft. long, covering and protecting the entrance to Erie Basin and Black Rock Harbor and the lake front between the New York State structures known as the Erie Basin breakwater and the Bird Island pier. Its construction was authorized by the River and Harbor Act of March 3d, 1899. See project marked *M* on Plate V.

The "Old" Breakwater.

The location of this structure, now locally known as the Old Breakwater is shown on Plate V. Its northern extremity is a

point almost due west 2 500 ft. from the lighthouse on the west end of the South pier. It extends thence southeast for 7 609 ft., parallel with and about 2 900 ft. from the shore, and in water ranging from 22 to 30 ft. in depth. As it now stands, it consists of a substructure, of separate timber cribs filled with stone, of a uniform length of 50 ft., with their tops sunk approximately to mean lake level.

At the northern end, for 2 926 ft., the cribs are 34 ft. wide; this is followed by a section of 1 968 ft., in which the cribs are 38 ft. wide, and those of the remaining section, 2 715 ft. long, at the southern end, are 36 ft. wide.

This entire substructure was originally surmounted by a continuous superstructure of timber cribwork filled with stone, and of the same width as the substructure. In the beginning, this superstructure was only 8 ft. high, but later, a 4-ft. parapet, 12 ft. wide, was added on the lake side.

The construction was begun on June 7th, 1869, when the first crib was sunk. The last one was placed in 1893, the timber crib superstructure being finished on October 13th, more than twenty-four years having been occupied in the original construction.

Owing to the long period occupied in the building, it became necessary, long before the completion of the project, to make extensive repairs to the superstructure, due to the natural decay of the timber, and its consequent destruction by storms. For this reason the present superstructure consists of three types: A continuous timber cribwork filled with stone, a concrete monolithic structure, and a concrete-shell structure filled with rubble.

Many of the early harbors on Lakes Erie and Ontario were formed, as at Buffalo, by the building of parallel jetties, or piers, at the mouths of the larger streams, dependence being placed on floods to scour out the contracted channel between them. Later, as the draft of vessels increased, of course, sufficient depth of water had to be secured by dredging.

The favorite construction used in these works was timber cribs filled with stone. This method had many points in its favor, among which may be mentioned, especially, ease of construction and cheapness, and, in the case of the work under water, comparative permanency.

Regarding structures of this class, the following instructions, formulated by the Board of Engineers, and of which Lieutenant-Col-

onel James Kearney, Corps of Topographical Engineers, was President, in charge of Lake Harbors and Western Rivers, on April 27th, 1853, are of interest:

"The Board having under consideration the principles of construction to be employed in building the piers, jetties and breakwaters on the Lakes, are of opinion:

"*First.*—That these works ought to be founded on cribs of timber filled in with stones.

"*Second.*—That the ordinary length of each crib should be 30 to 50 ft.

"*Third.*—That the breadth of the piers, etc., ought not to be less than 20 ft., even in the shoalest water.

"*Fourth.*—That their average width ought at least to equal their total height from the foundation to the platform.

"*Fifth.*—That the platform or deck of the piers, etc., ought to rise 5 ft. above high-water level of the lake.

"*Sixth.*—That the side exposed to the waves ought to be vertical from top to bottom; and, as much as possible, free from angular projections or breaks.

"*Seventh.*—That the sheltered side ought to be vertical from the platform to 5 or 6 ft. below the low-water level, and thence downward it ought to have a slope or batter in grades of one-third to the natural foundation.

"*Eighth.*—That the timbers used in the work should be all squared or sawed.

"*Ninth.*—That the exposed face of the work ought to be covered with plank, well fitted and bolted to the timbers, and to extend if possible from the top to the natural foundation.

"*Tenth.*—That whenever it is dangerously exposed to floating ice, it should be defended by wrought-iron bars or straps well bolted to the timbers.

"*Eleventh.*—That the bottom of the cribs ought to be of grillage, the openings in which should be barely sufficient to allow the stone filling of the cribs to settle to the ground.

"*Twelfth.*—That the rip-rapping, if any, around the bottom of the cribs, ought not to rise so high as to be exposed sensibly to the waves.

"*Thirteenth.*—That the piles, which are used sometimes in the inside of the cribs, have not the effect to keep them permanently in place or plumb.

"*Fourteenth.*—That the cribs beneath the water surface should be of the same width as the piers or breakwaters, and may be of different lengths. Above water the cribwork should be constructed so as to preserve a break-joint connection throughout the whole length of the work.

"*Fifteenth.*—That due time having been allowed for the ultimate settlement of the structure; the upper part of it should be replaced by masonry, namely, from a point so low that the timbers of the foundation may not be liable to come in contact with the atmosphere.

"The Board reserves for a future report the question of the dimensions of the timbers, the method of framing, or the carpentry, of the cribs, the filling in with stone, and the setting of the cribs into place.

"To enable the Board to arrive at satisfactory opinions upon points now referred to, and enter into the details which ought to characterize such a report, it will be necessary for the members to examine for themselves the works most worthy of notice, or those that are thought to be the most successful examples of cribwork on the Lakes, as well as those that are defective.

"The Board would thus be enabled to judge more satisfactorily as to the causes (arising either from inherent defects or accidents) that led to their failure; and, on the other hand, the causes that may have contributed to the efficiency of those that have succeeded."

When, in 1867, General Cram submitted a project for building a detached breakwater, which was to form an outer harbor, there was, as a matter of course, some discussion of the proposed plans, especially as to the proper cross-section of the cribs, and of their details.

The matter was referred to a Board of Engineers, convened in 1868, which recommended that the structure be built of cribwork filled with stone and decked over with plank, to have a rectangular cross-section; the width in 29½ ft. of water to be 34 ft., and in no case less than the total height from the bed of the lake to the top of the deck, and the top to be 5 ft. above the highest water known.

The location of the work and the construction details having been decided, the contract for the northern end was let to Messrs. Bailey and Denny in 1868.

The cribs were 34 x 50 ft., and 23 ft. deep, which was also the depth of the water, the cribs being set directly upon the natural bottom, without any preparation. With the exception of the top courses, which were pine, the cribs were of 12-in. square hemlock, duly framed, and fastened with 1¼-in. square iron drift-bolts 45 ins. long. Each crib contained about 110 000 ft., B. M., of timber, 11 tons of iron, and 280 cords of stone. The superstructure was of pine, and was built 8 ft. above the water level.

As the lake bottom of the area adjacent to the north end of the breakwater is generally rocky, overlaid with a thin layer of sand, no apprehension seems to have been felt in placing the cribs directly

thereon; in fact, it is doubtful if any extended examination of the bottom, along the line of the proposed work, was made or considered necessary at the time.

During the summer of 1872 a change in the character of the lake bottom was noticed. The hard gravel bottom became thinner and an underlying stratum of soft clay came near the surface. Much difficulty was found in keeping in position cribs sunk upon this yielding foundation. They sank as much as 4 ft. in 24 hours. In the following winter the cribs sank as much as 8 ft., the settlement being very uneven. The cribs were also displaced sideways to a greater or less extent, varying from 4 to 24 ft., but their integrity did not seem to be disturbed. The damage was repaired by sinking on top of the disorganized work a repair crib of irregular shape, from 4 to 24 ft. wide, 19 to 6 ft. high, and 315 ft. long. This was fastened to the old cribs by two iron rods, running through from side to side, and by piles driven in the corners to bed-rock.

In 1873 the mode of construction was changed. An artificial foundation was made by spreading over the lake bottom a layer of rubble, which, by its weight, settled into the soft clay. It was raised about 4 ft. above the lake bottom, and extended 25 ft. exterior to the lake face of the cribs, and 15 ft. on the harbor side. On this foundation two cribs were placed, each 34 x 50 ft. and 22 ft. deep. A superstructure was built over them and the displaced work of the preceding year.

On December 4th, 1873, a gale displaced these two cribs, wrecked 150 ft. of superstructure and damaged all the superstructure built in 1872–73. The 315 ft. of 1872 settled still more, and with great irregularity. The prepared foundation of broken stone sank through the soft clay so completely as hardly to be found by sounding. It was very evident that the prolongation of the line of that part of the breakwater already completed passed through an area of soft bottom which offered great difficulties in foundation.

Under the direction of Mr. (now Dr.) George E. Fell, the bed of the lake along the line of the extension was examined by drivings made from a scow provided with special machinery, and was found to consist in great part of a soft blue clay, into which the cribs might be expected to settle considerably. To prevent this settlement, a Board of Engineers, convened in the spring and again in the fall of 1874,

recommended an artificial foundation, to be made by dredging a cut in the bottom of the lake on the prolongation of the line of the breakwater, making this cut 50 ft. wide at the bottom, carrying it down to bed-rock, unless sufficiently firm clay, sand or gravel were found at a less depth; the side slopes of the cut to be as steep as the material would stand, and the cut, as fast as made, to be filled with gravel to the level of 3 ft. below the mud bottom, which level was to be reached by a covering of 3 ft. of rubble, having a width of 50 ft. at top. The Board supposed that this foundation would only be necessary for about 2 500 ft. from the terminus of the completed work, and for that distance estimated the average depth of dredging at 12 ft., at a cost not to exceed $150 000 (see Fig. 3). The Board also recommended that the

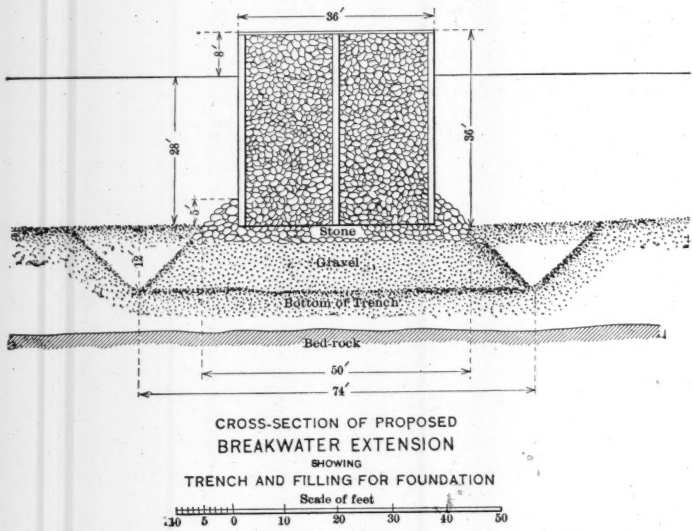


FIG. 3.

length of the breakwater be increased 3 600 ft., making its ultimate length 7 600 ft., this to make a large still-water harbor, as well as to cover a long shore line which would be available for docks.

The work of excavating this trench was commenced during the summer of 1875. At the beginning the depth of soft clay excavated was from 8 to 10 ft., but in 1878–79 it increased to from 19 to 22 ft.,

which delayed the work, the water being about 30 ft. and the bottom of the trench therefore about 49 to 52 ft. deep. Later, the depth decreased, and in 1882 this stratum pinched out entirely. Borings taken southward from this point for 1 500 ft. indicated that the bottom was composed of boulders and gravel, and gravel mixed with hard clay. It is stated that a pointed rod could not be driven into this material more than 4 ft., therefore there was no further apprehension as to future settlement.

The length of unstable foundation which required dredging and back-filling with gravel was about 2 500 ft. It underlies that portion of the breakwater built between 1874 and 1882.

As previously stated, the height of the superstructure, as originally constructed, was only 8 ft. above lake level. The length of this type of structure was 4 891 ft. It is a well-known fact that in heavy gales from the west Lake Erie rises 5 or 6 ft. and more, above ordinary low water, which would leave only 2 to 3 ft. of the breakwater above the surface. The effect of this high water is to create a heavy sea inside the breakwater. A gale of this kind occurred on January 3d and 4th, 1884, which did great damage to the superstructure for 800 ft. southward from a point 2 000 ft. from the north end, and a large part had to be rebuilt. To remedy the disturbance caused by the high seas, a parapet, 6 ft. high and 12 ft. wide, was added in the reconstruction, and all the work (2 718 ft.) constructed after 1884 was built after this plan, except with occasional and slight modifications as to height.

In the timber cribwork, generally, hemlock was used for the under-water work, except for the few top courses, between "wind and water," where pine (and, in a few cases, oak) was substituted. The superstructure was of pine, with the exception of the stringers, which were usually oak. The sides and ends of the cribs are of 12 x 12-in. timbers, built up solid. The interior cross- and longitudinal- ties are of the same dimensions, and are laid in "cob-house" style. At first, all timbers were fastened with square iron drift-bolts; later, round iron was substituted. The later cribs were built of two parts. The lower was a grillage, the timbers of which were fastened with screw-bolts. In the work above the grillage, drift-bolts were used. The superstructure is similar in design, except that, usually, the interior ties are laid close, forming solid bulkheads. The decking, of 3-in. plank, is laid at right angles to the axis of the work.

The work was paid for at unit prices for the various classes of material. In the beginning, a price was paid for the timber, and another price for framing. It was also the practice, at times, to let the cribwork and the bolts and spikes separately.

One prominent feature of some of the cribwork of the old breakwater was the abnormal length of some of the timbers used. In the grillage, not less than thirteen pieces of timber, each 50 ft. long, were used. In the hemlock portion of the cribs there was an extremely large number of 36-ft. pieces, aggregating nearly 70% of the total timber. This was on account of making the end walls and cross-ties in one piece. In the top part of the cribs and in the superstructure more than 30% of 36-ft. pine timbers and about 65% of oak timbers were used.

No figures, giving the total cost of the original timber-crib breakwater, are available. The Board of Engineers which determined the plans of this breakwater (March 27th, 1868), estimated the cost of a 100-ft. section, with cribs 34 x 50 ft., but depth not given, as in Table No. 1.

TABLE No. 1.—Estimated Cost of 100-Foot Section of Timber-Crib Breakwater.

12-in. timber	11 406 cu. ft.	@ $0.24	$2 737.44
Plank	5 580 ft. B. M.	" 20.00	111.60
Square bolts	14 335 lbs.	" 0.03⅞	555.58
Spikes	350 lbs.	" 0.05½	19.25
Rubble	1 750 cu. yds.	" 1.50	2 625.00
Framing	11 871 cu. ft.	" 0.12	1 424.52
Contingencies	10%		747.40
Total cost of 50-ft. crib			$8 220.79
" " " 100-ft. "			$16 441.58

In 1873, the theoretical cost (with no allowances for settling or contingencies) of 50 ft. of breakwater was estimated as in Table No. 2.

TABLE No. 2.—Theoretical Cost of Breakwater.

Timber and lumber	139 875 ft. B. M.	@ $20.00	$2 797.50
Framing	11 891 lin. ft.	" 0.12	1 426.92
Joist and plank, fitting, etc.	2 876 ft. B. M.	" 7.00	20.13
Drift-bolts	32 197 lbs.	" 0.03⅞	1 247.63
Spikes	680 lbs.	" 0.05½	37.40
Stone	435 cords.	" 6.85	2 979.75
Total			$8 509.33
Cost per linear foot			170.00

TABLE No. 3.—Cost of Timber-Crib Breakwater, per Linear Foot, for Different Years.

Old Breakwater, commenced on June 7th, 1869, and completed on October 13th, 1893.

Total length = 7608.7 ft.

Engineer officers in charge, from 1837 to 1903.	Contractors.	Date of contract, and of completion of work.	Location of work between stations (Station 0 at north end.)	Length of work, in feet. Individual contracts.	Cribs. No.	Cribs. Dimensions, in feet. Length.	Width.	Depth.	Cost of Work. Total.	Per linear foot.
Capt. W. G. Williams	Bailey and Denny	January 20, 1868								
Capt. I. C. Woodruff	D. E. Bailey	August 22, 1870								
Col. William Turnbull	D. E. Bailey	October 20, 1870								
Col. J. D. Graham	D. E. Bailey	June 28, 1871	0 to 29 + 24.0	2924.0	58	50	34	23		$230.00
Capt. John A. Tardy	D. E. Bailey	July 5, 1872								
Col. T. J. Cram	D. E. Bailey	April 24, 1873								
Major F. Harwood	D. E. Bailey	May 13, 1875								
Lt.-Col. C. E. Blunt	Ambrose Clark	July 25, 1877								
		September 9, 1878	29 + 24.0 to 30 + 76.0	150.0	3	50	38	28		160.00
Major Walter McFarland	Franklin B. Colton	October 29, 1878								
		December 1, 1879	30 + 76.0 to 34 + 29.0	353.0	7	50	38	28		184.00
Lt.-Col. H. M. Robert	D. E. Bailey	January 27, 1880								
		August, 1881	34 + 29.0 to 39 + 33.0	504.0	10	50	38	28	$104 259.31	*206.86
Capt. E. Maguire	D. E. Bailey	September 25, 1880								
		November, 1881	39 + 33.0 to 44 + 37.0	504.0	10	50	38	28	104 225.78	*206.80
Capt. Carl F. Palfrey	Hingston and Woods	May 27, 1881			8	50	38	28		
		November, 1882	44 + 37.0 to 48 + 91.0	454.0	1	50	38	25		160.00
Capt. F. A. Mahan	Charles E. Williams	February 21, 1884			5	50	36	24	89 783.58	111.40
		October 29, 1884	48 + 91.0 to 56 + 96.9	805.9	11	50	36	22		
Major Amos Stickney	Ira Farnsworth	October 4, 1884								
		October 10, 1885	56 + 96.9 to 63 + 49.8	652.9	13	50	36	22	64 173.44	98.29
Major E. H. Ruffner	J. B. Donnelly	April 17, 1891								
		November 9, 1891	63 + 49.8 to 68 + 02.4	452.6	9	50	36	22	60 189.91	*132.99
Major Thos. W. Symons	J. J. Churchyard	November 28, 1892								
		October 13, 1893	68 + 02.4 to 76 + 08.7	806.3	16	50	36	22	112 288.12	140.02
Major Theo. A. Bingham										

*Including inspection.

Table No. 3 shows the cost of the timber-crib breakwater per linear foot, for different years. In this table it will be noted that there is quite a variation in cost. The low cost for 1877–78 was owing to the fall in prices of material and labor. In 1879–80–81, the depth of soft clay, excavated for the foundation, and refilling with gravel, was greater than under any other contract, hence the increased cost. In 1884 and afterward no trench excavation or backfilling was necessary, the natural bottom of the lake being hard enough to sustain the work. The rates in Table No. 3 include all expenses of every kind, the net cost of the workmanship being 6 to 10% less than those rates, according to the amount done.

As a rough guess, it may be stated that the total cost of the breakwater was, approximately, $1 500 000.

Concrete Superstructure.—Great damage was done to the structure by the gale of October 14th, 1886, the heaviest experienced since November 16th, 1869. The wind was from the southwest, and reached, at one time, a velocity of 70 miles an hour. The level of the lake was raised 7.3 ft. above standard low water. For a length of 1 910 ft., beginning at the north end, almost the entire harbor face of the breakwater was pushed out of place. The ends of the cross-ties had become so rotten that in many cases the wood of the dovetails stripped off. The ice of the following winter also did more or less damage, especially to the cross-walls or bulkheads of the parapet (first built in 1884), which were partly laid up "cob-house" fashion, the ice being piled up 30 ft., or more, in height. In many places, the lake face of the breakwater was also crushed in by the ice. The dilapidated condition in which the north end of the breakwater was left after this storm, made it evident that the reconstruction of the superstructure was an absolute necessity. On December 15th, 1886, a plan for a new superstructure, of concrete, faced with stone, was submitted by Captain F. A. Mahan. Some changes in the same were made, it being ordered to build 500 ft. with stone facing and 250 ft. with a facing of Portland-cement concrete, the work to be done by day's labor and the materials to be obtained under contract.

The approved plan, as finally adopted, was as follows:

1.—The timber cribs of the foundation were to be cut down to 2 ft. below mean low water;

2.—On the razed cribs was to be built a masonry foundation, 5 ft.

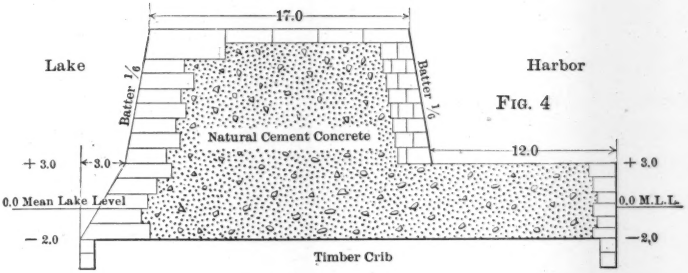


FIG. 4

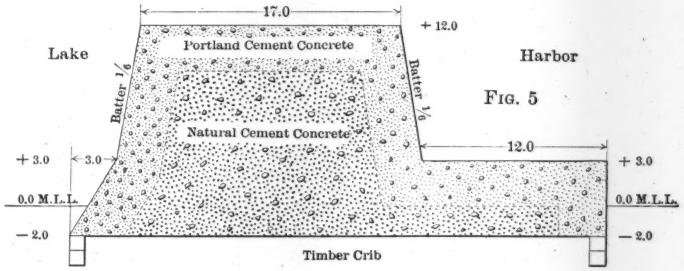


FIG. 5

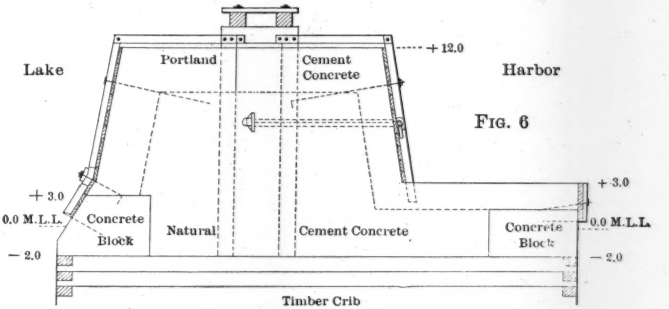


FIG. 6

high, of the same width as the cribs, vertical on the harbor side and with a slope of 5 on 3 on the lake side;

3.—On this structure was to be built a parapet, 20 ft. wide at the bottom, 17 ft. wide at the top, and 9 ft. high, the slopes of both the harbor and lake sides being 6 on 1; the lower edge of the lake side of the parapet to coincide with the upper edge of the lake side of the foundation.

The new superstructure was to be built in two ways:

1.—The inside mass of natural-cement concrete to be covered with stone masonry of varying thickness; on the lake face, 3 ft.; on top, 2 ft. for 5 ft. back from the lake face; the remainder of the top, 1 ft.; the harbor face of the foundation, 2.5 ft.; the top face of the foundation, for a width of 11.5 ft., to be uncovered.

2.—The concrete mass was to be covered on all sides with a layer of Portland-cement concrete, 3 ft. thick.

The work was commenced, about June, 1887, by tearing out the timber superstructure and preparing the tops of the cribs to receive the concrete.

The laying of the concrete was begun on July 30th, 1887, and closed for the season on November 14th. After August 11th, work was prosecuted night and day, and, after September 4th, also on Sundays. See Figs. 4 and 5.

Details of Construction.—The plant consisted of a derrick scow 92½ x 20 ft., drawing 2 ft. when loaded. The derrick was fitted with a long boom for light loads and a short one for heavy loads. It was operated by a single-cylinder double-drum engine. Near the stem there was a small blacksmith shop. There were also three scows, each 95½ x 17 ft., and drawing 5 ft. when loaded. One scow was fitted with a concrete mixer at one end, the mixer being a 4-ft. hollow cube of boiler iron, driven by a single-cylinder engine. At the other end there was a platform on which the cement paste was mixed. A track connected this platform with the mixer. There was also another scow, 55½ x 20 ft., drawing 3¾ ft., which, with two of the larger ones, was used for the transportation of material. A small derrick scow was used by the stone masons, and a large flat scow, of about 300 tons capacity, was used for the storage of sand and gravel.

The tools were of the usual kinds. The rammers were of cast iron, with wrought-iron handles. The base was 5 ins. on an edge at

the bottom, 3 ins. at the top, and 6 ins. high. The wheel-barrows carried boxes holding 2 cu. ft. each.

The concrete forms were of wood, with 4 x 6-in. posts or braces, attached to the cribwork timbers by wrought-iron knees. The sheeting was of 2-in. plank spiked to the posts.

Execution of the Work.—The derrick boat was placed broadside along the harbor face of the breakwater. In front of this boat, and head on, lay the mixer boat, and immediately in the rear was the large float loaded with sand and pebbles. On one side of this float was placed the emergency stone scow, and on the other side the contractor's stone scow. When the boats were all in position, runways were laid from the mixer scow to the various vessels, so that the materials could be wheeled up conveniently. The arrangement of the scows is shown in Fig. 7.

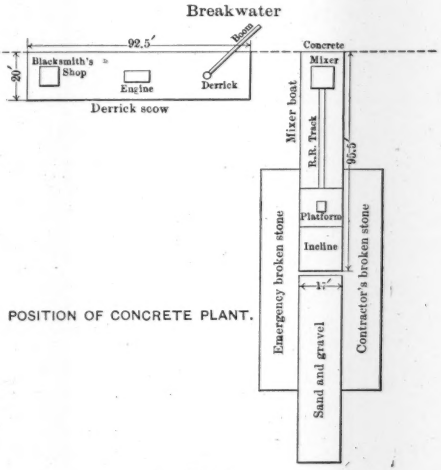


FIG. 7.

The concrete was made as follows: The cement (2 cu. ft.) was mixed into a rough paste on the platform. The other ingredients, in the proper proportions, were then wheeled to the platform and dumped through a hole in the same into a large box standing on a railroad truck beneath, in the following order: First the pebbles, then the sand, and then the broken stone. The cement paste was thrown in with the sand. The loaded box was then run out, raised by the derrick, and dumped into the mixer, the box was returned and the operation repeated.

After from five to ten revolutions of the mixer, the concrete was emptied into another box, and placed in the work by the derrick. It was then spread in layers of 6 to 8 ins. and rammed thoroughly.

With full gangs, fourteen batches per hour could be placed. The average for the entire season was about seven batches per hour. With

the larger number of batches, twelve men were required for ramming alone.

The construction of the stone face was very simple, and differed in no respect from the ordinary methods of constructing masonry.

Difficulties Encountered.—The greatest difficulty was caused by the instability of the boat. The management of the boxes of concrete was exceedingly troublesome. Comparatively small oscillations of the boat amplified greatly the movement at the end of the boom. Difficulty was also found in placing the concrete at the salient angles at the bottom of the structure. It was washed out by the swell caused by passing boats, or by the wind, and the angles then crumbled. Every way that the ingenuity of the force could devise was adopted to try to prevent this wash until the cement could set. Not one was successful. The best results were obtained by the use of tarred paper and crash, with which the effect of the wash was less than by any other method.

The weather, as a rule, was favorable, with the exception of two gales, which did some damage, washing out some of the interior concrete. All the work was done by hired labor, the plant being owned by the United States Government.

Cost of the Work.—The Portland cement used was the "Gibbs," from London, England. The contract price was $2.09 per barrel, delivered in Buffalo by canal. A little came by rail, which made the average price 2.16\frac{8}{10}$. Some additional cement needed cost $2.57 per barrel.

The natural cement was furnished by F. O. Norton, of New York, at $1.20 per barrel, delivered in Buffalo by canal. Sand cost 80 cents per cubic yard, and pebbles $2 per cubic yard. Broken stone, furnished by contract, cost $1.84 per cubic yard. Owing to the inability of the contractors to supply broken stone promptly at all times, some was broken by hired labor at a cost of 3.25\frac{3}{4}$ per cubic yard.

The cost given has reference to the work as it stood on April 1st, 1888, being practically that done during the season of 1887.

The quantity of concrete made was as follows:

Portland-cement concrete	2 937.71	cu. yds.
Natural-cement concrete	2 149.58	" "
Total	5 087.29	" "
Less large stone in same	300.00	" "
Net quantity of concrete	4 787.29	" "

This quantity was made in 5 797 batches, giving 0.8258 cu. yd. to the batch. The quantities required to make a cubic yard of concrete are shown in Table No. 4.

TABLE No. 4.

	Concrete.	
	Portland cement.	Natural cement.
Cement, cubic feet	3.288	2.411
Sand, " "	5.021	5.021
Pebbles, " "	14.805	14.805
Broken stone, cubic feet	14.473	14.473
Total	37.587	36.710

In the statement of cost in Table No. 5 the items of material and labor only are included:

TABLE No. 5.

				Concrete.	
				Portland cement.	Natural cement.
Portland cement	2 383	bbls. at	$2.168 =	$5 166.34	.....
" "	193	"	2.57 =	496.01	.....
Natural "	1 383	"	1.20 =	.....	$1 659.60
Sand	946	cu. yds. at	0.80 =	437.02	319.78
Pebbles	2 789.6	"	2.00 =	3 221.77	2 357.43
Broken stone	1 778	"	1.84 =	1 841.20	1 347.24
" "	949	"	3.25¾ =	1 785.10	1 306.19
Labor				6 952.43	5 087.22
Total				$19 899.87	$12 077.46
Cubic yards in place				2 937.71	2 149.58
Cost per cubic yard				$6.774	$5.62

The cost of the 250-ft. section was $73.39 per running foot.

Owing to the excessive cost of the masonry-faced section (200 ft. long) it was recommended that this method of construction be discontinued.

Concrete Blocks.—As it was almost impossible to preserve the integrity of the angles at the bottom of the profile when building the concrete in place, authority was given, in February, 1888, to use large concrete blocks at the bottom of the work. The blocks for the lake face were 6 ft. long on the bottom, 4 ft. 2 ins. wide, and 4 ft. high, and were set as headers, the end toward the lake rising vertically for 1 ft.

and then having a batter of 5 vertical to 3 base, the opposite end being vertical. The blocks on the harbor face were rectangular parallelopipedons, 6 ft. long, 4 ft. 2 ins. wide, 3 ft. 6½ ins. high, and contained the same volume as the blocks on the lake face, namely, 88.75 cu. ft. or 3.287 cu. yds. See Fig. 6, page 91; also Fig. 9, page 101.

The first block was made on August 6th, 1888, and at the close of actual operations, on November 28th, 1888, 625 had been made; of these four were damaged so as to be useless. The blocks were made in wooden moulds, the grooves for the reception of the hoisting chains being formed by battens. Forty-two of these moulds were made, costing $1 675.99, or $39.90½ each.

The blocks were made on the south pier, behind the parapets of the old breakwater. The concrete was hand-mixed in the following proportions: ½ bbl. of Portland cement, 2½ cu. ft. of sand, 7½ cu. ft. of pebbles, 7½ cu. ft. of broken stone.

These quantities made a layer of concrete a trifle more than 6 ins. thick. The face of the lake-face blocks was made of rather finer materials than the inside. This finer concrete was about 6 ins. thick, 6 to 8 ins. wide, and well rammed. The coarser concrete was spread just behind it, before the face had set, and, being properly rammed, gave a thorough bond between the face and the back. The block was built up in this way in successive layers. In the top layer irregular masses of stone were set so as to project 6 or 8 ins. above the top of the block when finished. These stones served as dowels to insure a firm bond with the superstructure. One gang of men, on several occasions, made fourteen blocks in 10½ hours. This, at 3.287 cu. yds. to the block, gives 46 cu. yds. per day, or 4.37 cu. yds. per hour.

The total volume of the blocks is 2 054.37 cu. yds. The materials used were:

Sand	364.55 cu. yds.
Pebbles	1 175.10 " "
Broken stone	972.22 " "
Cement	258.09 " "
Total volume of loose materials	2 769.96 " "
Total volume of blocks	2 054.37 " "
Shrinkage	715.59 " "

or 25.8 per cent.

The disparity in sand and other materials was caused by many of the blocks requiring to be placed on a special bed while under construction. The excess of pebbles over broken stone was caused by using the finer material for the lake face, so as to expose a smoother and more solid surface to the waves.

The cost of the materials for the blocks was:

Cement	$4 464.98	
Sand	291.64	
Broken stone	1 701.39	
Pebbles	2 350.20	
		$8 808.21
Superintendence	$476.04	
Labor	3 908.55	
		4 384.59
Tools, supplies, tug hire, boats, repairs, etc		4 085.04
Moulds		1 675.99
Total cost of making 625 blocks		$18 953.83

This gives, as the cost per block:

Materials	$14.093
Superintendence and labor	7.015
Miscellaneous	6.536
Moulds	2.682
Total cost of a block ready to set	$30.326

or $9.226 per cubic yard.

The following prices were paid for the materials:

Portland cement	$2.45	per barrel.
Sand	0.80	per cubic yard.
Pebbles	2.00	" " "
Broken stone	1.75	" " "

The handling and setting of the blocks was very simple, and was accomplished by a floating derrick, lashed alongside of which was a scow, connected by 12-in. timbers to steady it. This made a very stable arrangement.

In the cost of setting the blocks is included the cost of recovering

(by means of a diver) 349 blocks which were washed overboard by severe gales. Of these, twenty-four were damaged so much that they were unavailable for use in the faces of the structure, and had to be used in the interior.

Cost of making blocks	$18 953.83
Cost of setting blocks	4 239.84
Total cost of blocks	$23 193.67

This amount, apportioned among the 621 blocks actually used, makes the cost $37.35 each, or $11.29 per cubic yard.

The wages paid to the various artisans were as follows:

Master laborers	25, 27½ and 30 cents per hour.
Laborers	17½ and 20 " " "
Carpenters	25, 27½ and 30 " " "
Blacksmiths	17½ " " "
Enginemen	25 and 30 " " "

Tearing out the Old Timber Superstructure.—Various storms, beginning with that of October 14th, 1886, had done more or less damage to the old timber superstructure. In some places the harbor wall of timber had been carried away to the water's edge, and the stone in the pockets had been washed out. In other places the stone had been dug out of the pockets and the timber left standing. Nearly one-third of the timber had been torn off. To remove the timber which remained, various methods were tried, such as pulling up sticks by means of a derrick boat, sawing and splitting the timber away from the drift-bolts, pulling the drift-bolts with a heavy shackle bar, etc. The most satisfactory and expeditious method, however, was that of drawing the bolts partly with a hydraulic jack and finally with a shackle bar. By this method the timber was uninjured, except for the small cut made at each of the drift-bolts to allow the jack to take hold.

After the bolts were removed, the timbers were readily taken apart, and many of them were used for trestling, for scaffolding, and for stringers in connection with the railway track used in transporting the concrete to the work.

PLATFORM FOR MIXING CONCRETE.

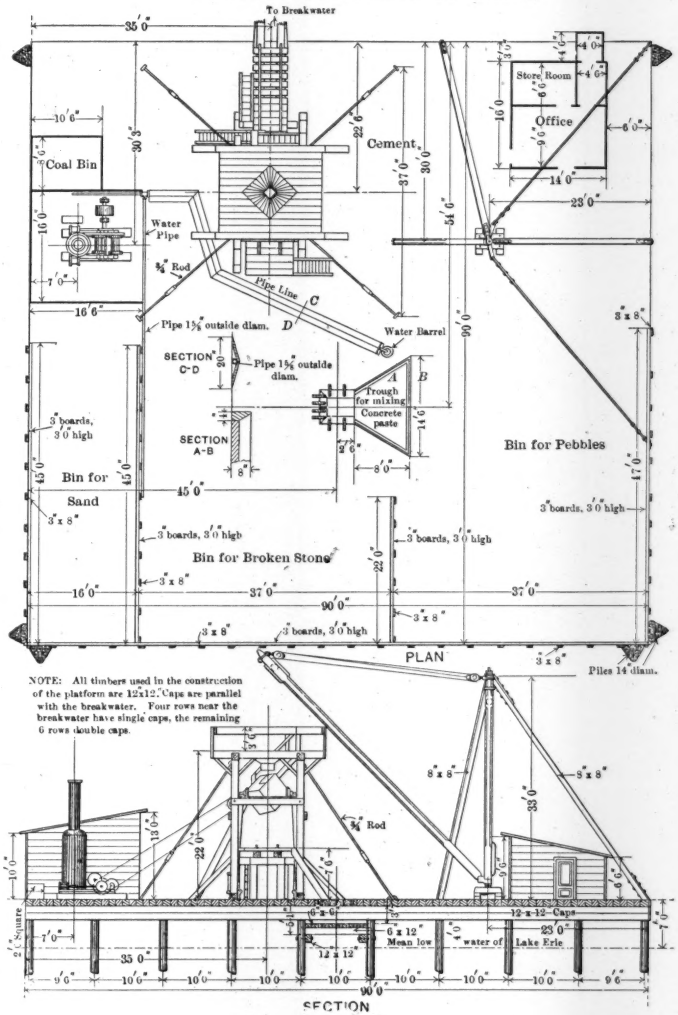


FIG. 8.

The cost of tearing out the old superstructure was as follows:

Superintendence	$287.07
Labor	5 110.34
Miscellaneous, including stone removed by contract, tools, tug hire, boats, etc	2 509.54
Total cost	$7 906.95

The length of the structure torn down was 1 460 ft., making the cost per linear foot 5.41\frac{6}{10}$.

Concrete in Place.—On account of the difficulties encountered during the season of 1887 in making and depositing concrete with a floating plant, the question of a different plant to be used in continuing the work (the construction of 1 460 ft. during 1889), was considered by a Board of Engineers in December, 1888. This Board recommended a traveling platform, designed to carry on a lower deck the mixer, boiler, engine, coal bins, cement, tools and other appliances, and, on an upper deck, the bins containing sand and broken stone.

The plan suggested was to set all the blocks, and fill in behind them with concrete to a depth of 2 or 4 ft., then, working backward, build up to a level of 7 ft., then, working forward, finish to the required height of 12 ft.

It was also contemplated to utilize, for the storage of materials and the making of concrete, that portion of the breakwater with which there had been trouble on account of settlement and displacement of the cribs, and which had a width of from 30 to 45 ft.

On January 9th and 10th, 1889, occurred the most furious wind storm known in Buffalo. From 4 A. M. until noon of January 10th the hourly velocity of the wind was never less than 61 miles. From 8 to 9 A. M. it averaged 78 miles. During this storm 346 of the concrete blocks, weighing about $7\frac{1}{2}$ tons each, which were lying on the banquettes of the breakwater and sheltered by the parapet, were washed overboard. As previously stated, much damage was done to the 1 460-ft. section upon which the concrete superstructure was to be built during the season of 1889.

After the effects of the storm were fully known, the Board of Engineers, in March, 1889, decided that, in view of the changed conditions, temporary fixed platforms would be more expeditious than the traveling platform previously recommended; also, that it would

not be advisable to use the banquette deck of the breakwater for the storage of materials and machinery.

In accordance with this decision, two platforms, each 90 ft. square, were built close to the harbor side of the breakwater, one at the quarter point and the other at the three-quarter point of the work. They were constructed of piles, capped with 12 x 12-in. timbers and

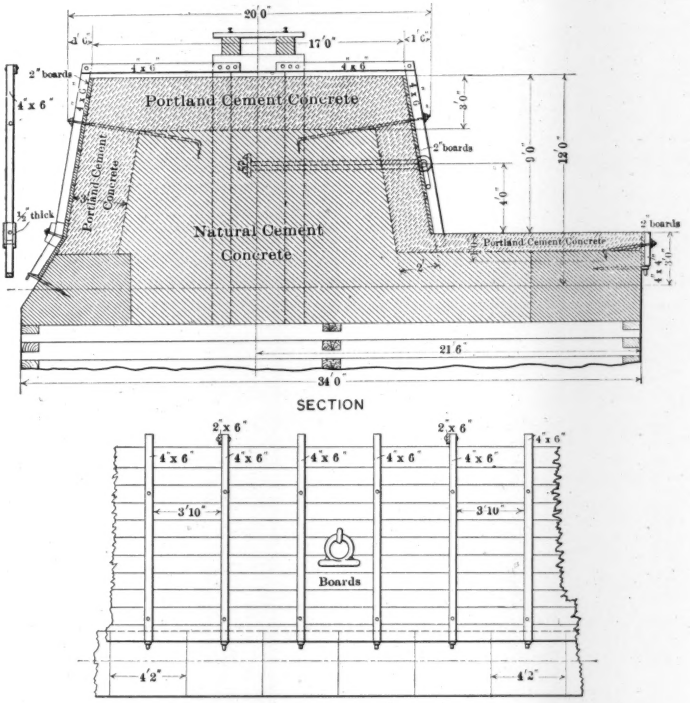


FIG. 9.

decked over. On these platforms were located the concrete mixer, engine and boiler, derrick, steam pump, and bins for sand, pebbles and broken stone. See Fig. 8. The platforms were connected by proper turnouts, with a temporary single-track railway, with trestle bents, 10 ft. apart, and running the entire length of the work. See Fig. 9, also Fig. 6.

The formula for the concrete was: 1 bbl. of cement, 5 cu. ft. of sand, 15 cu. ft. of pebbles, and 15 cu. ft. of broken stone.

To meet these requirements, hand carts of the capacity needed to carry the exact quantity of each sort of material were built. In addition, there were charging boxes, each of 40 cu. ft. capacity, and these were lowered to a sub-platform through a hole in the deck of the platform, their tops being level with the deck for convenience in charging. The cement was made into a rough paste in a special trough. In charging, the pebbles were dumped first, then the paste was shoveled in at the same time that the proportions of sand and broken stone were served. An additional quantity of water was poured in, and the charge hoisted to the mixer. From the mixer the concrete dropped into a car, which carried it to the work for deposit.

With these arrangements, the quantity of concrete from one mixer has been as much as 225 cu. yds. in 10½ hours.

The volume of the new superstructure is as follows:

Concrete blocks	1 962.34	cu. yds.
Masonry	228.00	" "
Large stones	679.30	" "
Damaged blocks	56.78	" "
Concrete in place	14 586.78	" "
Total	17 513.20	" "

The number of batches of concrete was 14 624, from which it is found that each batch, rammed in place, had a volume of a little less than a cubic yard.

The materials were mainly supplied by contract, at the following prices:

Portland cement	$2.45 per barrel.
Natural cement	0.82 " "
Sand	0.80 per cubic yard.
Pebbles	1.75 " " "
Broken stone	2.00 " " "

The Portland cement was made by Burham and Co., of England. The natural cement was made by the Union Akron Cement Co., in the

neighborhood of Buffalo. The sand came from the south shore of Lake Erie; the pebbles were brought from the Niagara River, near the International Bridge, and the broken stone came from Lockport.

There was also used in the work another English Portland cement, the Francis, at $2.45 per barrel, also Millen and Sons' domestic Portland, at $2.25, also some of F. O. Norton's natural cement, at $1.20.

One barrel of cement was used in each batch of concrete, and, as each batch made practically a cubic yard, the cost of the cement per cubic yard was the same as the price per barrel. The cost of the other materials in each batch was: Sand, 15 cents; pebbles, 97½ cents; broken stone, $1.11; a total of $2.23½.

Cost of the Concrete.—The elements of the cost of the concrete are, labor, material and miscellaneous expenses. Labor is divided into superintendence, or time of master workmen; and labor proper, or time of men acting under the guidance of the master workmen.

Superintendence	$1 426.41
Labor	20 851.73
Total	$22 278.14

The cost per cubic yard for superintendence and labor was $1.53. The largest labor item was the ramming, which was 25 per cent.

The cost of materials was:

Cement	$21 144.91	
Sand	2 193.60	
Pebbles	14 258.40	
Broken stone	16 232.64	
		$53 829.55
Material and labor for grouting and filling holes		274.82
" " " " large stone used in concrete		2 576.63
Masonry used instead of concrete blocks		5 587.40
Total		$62 268.40

The miscellaneous expenses were:

Cost of two platforms	$14 390.41
Derricks, machinery, pumps, rails, tug hire, etc.	29 647.53
Total	$44 037.94

The total cost of reconstructing 1 460 ft. of the superstructure of the breakwater was:

Tearing out old timber work		$7 906.95
Making and setting concrete blocks		23 193.67
Superstructure; concrete in place:		
Labor	$22 278.14	
Materials	53 829.55	
Extra work	8 438.85	
		84 546.54
Miscellaneous expenses		44 037.94
Total		$159 685.10

The cost was $109.37 per running foot, or $9.12 per cubic yard.

The value of the plant at the end of the season was taken to be $18 470. This, subtracted from the total, leaves $141 215.10; thus the net cost of the new superstructure was $96.72 per running foot, or $8.06 per cubic yard. The cost of the concrete in place, added to the cost of the platforms and the other miscellaneous items, gives:

Concrete in place (labor and material)		$76 382.51
Platforms		14 390.41
Miscellaneous		29 647.53
Total		$120 420.45
Deduct value of material and machinery on hand		18 470.00
Remainder		$101 950.45

Dividing this by 14 586.78, the cubic yards of concrete, gives $7.00 per cubic yard as the net cost of the concrete in place.

Sixty-eight days were required to place the concrete, thirty-eight days with one platform and thirty with two. This gives an average output of 214.51 cu. yds. per day.

After the decayed superstructure at the north end had been reconstructed, it was decided to replace another section, immediately south of that part which settled during 1872. This settled portion is known as the ice-breaker, and will be designated thus hereafter. The section adjoining the ice-breaker was to be 1 967.9 ft. long. During the season

of 1890, 950 large concrete blocks for the same were made. The River and Harbor Bill of September 19th, 1890, appropriated $200 000 for this work.

Before commencing the concrete superstructure, it was necessary to remove the old decayed timber superstructure, which was a work of no little magnitude, as it contained about 760 000 cu. ft. This work was commenced on April 9th, 1891, and by June 15th, 1891, the structure was about all removed to a plane 2 ft. below mean lake level. This removal necessitated the handling of 1 678 280 ft. of timber, 120 000 lbs. of iron, and 19 500 cu. yds. of stone. On June 8th, 1891, the setting of the concrete blocks was commenced, and at the end of the year (June 30th, 1891), 800 had been placed in position. The platforms on piles for storing material for concrete, one at either end of the work, and the concrete mixers were ready at about the same date. Trestlework had been erected along the line upon which the superstructure was to be built, and upon this two tracks were laid. The cars for carrying the concrete were moved by a wire cable operated by a steam engine.

On July 4th, 1891, a heavy gale washed from the breakwater 362 blocks, each weighing about 7 tons. The blocks were all recovered by a diver, the last one being raised on August 23d, 1891.

The laying of the concrete began on August 4th, and was practically finished on October 12th. An experimental top parapet, 300 ft. long and 5 ft. high, was begun on October 17th and finished on the 19th. All top dressing was finished on November 8th.

The machinery and methods of construction have been described, and it only remains to note that a great volume of concrete was laid quickly and successfully, the cable road being a perfect success, and previous experience having led to skill and promptness in the details of construction.

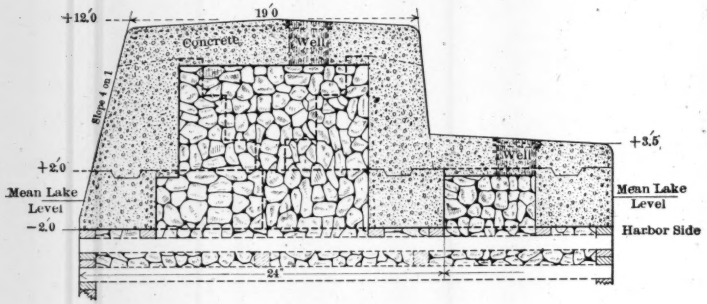
The length of the concrete superstructure built during the season was 1 967.9 ft. The volume of concrete was 25 969.28 cu. yds., or 132 cu. yds. per linear foot, the cost being $213 165.69, or $108.32 per running foot, or $8.21 per cubic yard. In 1889 this cost was $110.19 per running foot, or $9.19 per cubic yard. Experience and an improved arrangement of the machinery, and especially the moving of cars by cable instead of by hand, account for the decreased cost.

All the concrete work just described was done by hired labor, under the direction of the United States Engineer Officer in Charge.

In 1898 it was recommended by Major T. W. Symons that the ice-breaker should have the wood and stone superstructure replaced with concrete. This section is 1 015 ft. long, and was built in 1873 to 1877. The section concreted in 1889 adjoins it on the north, and that built in 1891 on the south. The delay in concreting it was on account of the insecurity of its foundation. This was made secure by dumping a large quantity of large rip-rap along the lake face of the structure.

Plans and specifications were prepared during the winter of 1898–99, and the work was let to the Buffalo Dredging Co. on May 31st, 1899. The design of the new work was a radical departure from that

PLAN FOR REPLACING DECAYED TIMBER SUPERSTRUCTURE OF A PORTION OF BREAKWATER WITH A NEW SUPERSTRUCTURE OF CONCRETE AND STONE



MINIMUM CROSS SECTION
HEAVY DOTTED LINES SHOW CROSS-WALL BLOCKS AND DIVISIONS.

The Maximum Cross section extends 32 ft. on the Harbor Side and 22 ft. on the Lake Side, making a total width of 78 ft.

Scale of Feet
0 5 10 15 20

FIG. 10.

previously executed, which was a monolithic structure. Instead, Major Symons adopted what may be called a shell construction, consisting of inner, outer and cross-walls, the interior space filled with rubble and the whole covered with a heavy parapet deck of concrete. The different walls were founded on large concrete blocks, with their bases laid 2 ft. below mean lake level. The construction is shown in Fig. 10.

The contractors began the erection of a concrete plant and preparation for a floating plant on June 5th, 1899, and commenced

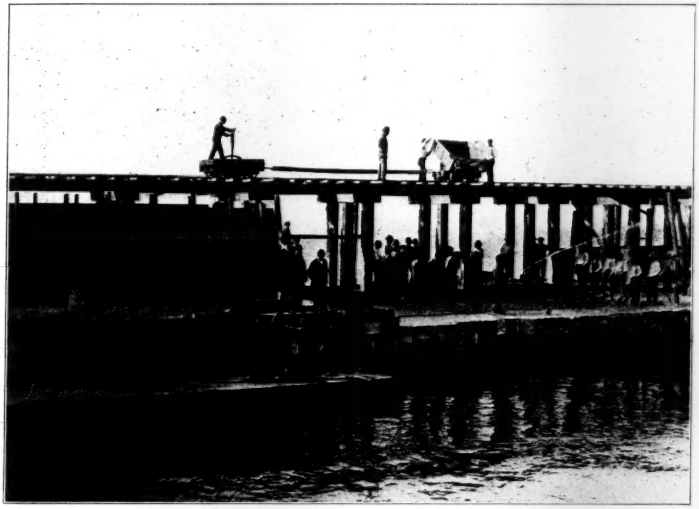


FIG. 1.—OLD BREAKWATER. BUILDING MONOLITHIC CONCRETE SUPERSTRUCTURE, 1891.

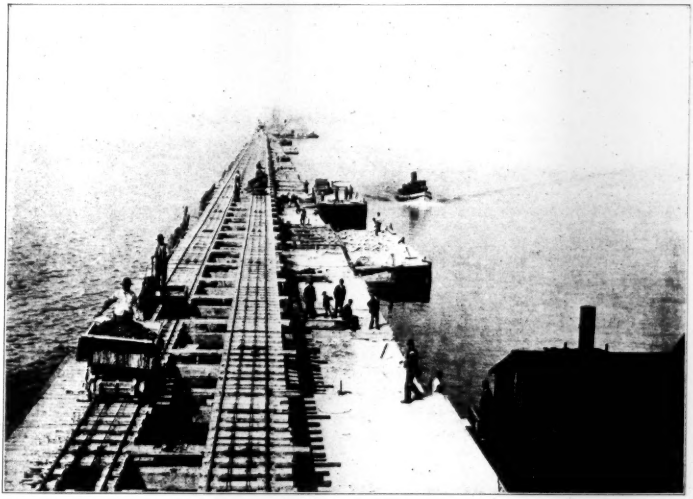


FIG. 2.—OLD BREAKWATER. BUILDING MONOLITHIC CONCRETE SUPERSTRUCTURE, 1891.

making concrete blocks on June 8th. They began to tear out the old superstructure on August 5th, and the first concrete block was set on the lake face at the south end on August 16th. The first concrete deposited in place was in the lake wall of Section 1, which was laid on August 30th and 31st.

American Portland cement was used, the brands being the Wayland and the Lehigh. The concrete in the blocks contained 1 part of cement, 2 parts of sand (of which 1 part was stone-crusher sand), 2 parts gravel (of which 1 part was granolithic) and 2 parts of broken stone. The exposed faces of the lake- and harbor- face blocks, for a thickness of from 3 to 6 ins., were made of granolithic concrete. The blocks were made on the north bank of Buffalo Creek, just above Louisiana Street, a dockage of about 400 ft. being used. A switch of the Erie Railroad extended into the yard, making the delivery of the material convenient. The cement for the blocks was stored in warehouses in the yard, that for the mass concrete was stored elsewhere.

The greater part of the broken stone, granolithic, and stone-crusher sand was used directly from the railroad cars, while the remainder, with the sand and gravel, was stored in piles at convenient distances from the mixer.

A 4-ft. cubical mixer was used. It revolved on a 4 in. hollow shaft through which water was supplied. About 1 cu. yd. of concrete was mixed at a time. The material was delivered to the mixer in a dump car, divided into proportional parts, to admit the different materials in the proper quantities. The cement and beach sand were shoveled together before being placed in the car. The car was then drawn by steam power up an incline to a platform above the mixer, from which it was dumped into the hopper. After being turned fifteen to eighteen times, the mass was dumped into another car and drawn by steam power up an incline to the level of the top of the moulds, from which point it was propelled by hand.

The concrete was deposited in the moulds in layers of about 6 ins., and each layer was thoroughly tamped before another was put on. Five or six blocks were usually under way at the same time. On the lake- and harbor- face blocks a 4-in. piece was set upon the face side, while the regular concrete was being dumped, leveled, and partially tamped. It was then drawn out and the space filled with granolithic concrete. The whole mass was then thoroughly tamped together.

Ten or eleven blocks were made in a 10-hour day when the plant was running with a complete force. The moulds were in four rows, parallel to the dock face and as near to it as possible, occupying a length of about 150 ft., so that the blocks could be handled by a floating derrick. There was room to make from 47 to 50 blocks before any were removed. To facilitate handling, from one to three eye-bolts, depending upon the size of the block, were incorporated in the top of each block. These eye-bolts were either of new iron, 1 in. square and 3 ft. long, or were 1¼-in. drift-bolts taken from the old timber superstructure. Each had a 4-in. cast-iron washer on the lower end. The eye-bars made the handling of the blocks very satisfactory.

The moulds were of 2-in. pine, planed and matched, and the frames of 6 x 8-in. timbers, all held together by iron tie-rods. At first, 12-in. timbers were used, but were found to be too heavy. Beginning with the season of 1900 the inner faces of the lake- and harbor- face blocks were made ½ in. shorter than the outer faces, in order to make better joints. Pieces of lath were set up between the ends of the moulds and the frames, so that the frames did not have to be rebuilt.

The blocks were allowed 48 hours to set, before removing the moulds, and two weeks before being moved. When ready to be moved, they were loaded on scows by a large floating derrick and towed to the breakwater, where they were unloaded and stored.

Removing the Old Superstructure, and Repairing and Leveling Cribs.—This work required the removal of the timber and stone of the old superstructure, the repairing of the cribs when necessary, the filling of the openings between the cribs, and the careful leveling of the timber and stone, to 2 ft. below mean lake level, to receive the concrete blocks.

A specially large and powerful derrick was used in tearing out the timber and setting the concrete blocks. An old dredge was rigged for this work, the old machinery giving ample power. The anchors or spuds were found to be valuable adjuncts when setting the blocks, as they prevented the swaying or rolling of the scow and derrick.

The removal of the timber was accomplished with little cutting of timber or bolts. A pair of hooks engaged one timber at a time, and the derrick lifted it out intact or broke it off. In several places the oak sheeting gave considerable trouble and had to be sawed and taken out in sections.

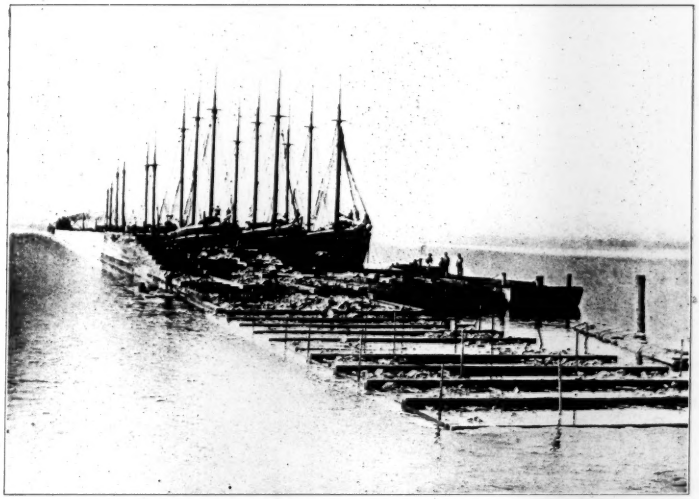


FIG. 1.—OLD BREAKWATER. BUILDING TIMBER SUPERSTRUCTURE ON CRIBS. 1891.

FIG. 2.—OLD BREAKWATER. SETTING CONCRETE BLOCKS IN INTERIOR. 1899.

During the season of 1899 a large part of the stone from the crib-work was stored on the banquette while the parapet was under construction, but in 1900 it was loaded on scows, taken to a storage yard, and there kept until needed to fill the new superstructure.

After the removal of the timber and stone to the required depth, seven cribs, uncovered during 1899, were found to be in fairly good condition. Up to June 1st, 1900, five more cribs had been uncovered, and were found to be in bad condition as to alignment. In one case the entire lake wall seemed to be gone. Piling was driven at this point, to correct the alignment. Several openings were found between the cribs, which were sufficiently large to require filling in.

The concrete blocks were set by the derrick used in tearing out the old timber. The lake-face, cross-parapet, and middle blocks were usually set in the order named, a section length being set before any concrete was moulded in place in the section. The setting of the harbor-face and cross-banquette blocks, as well as the moulding of the banquette deck, was usually left for rough-weather work.

Concrete in Place.—The concrete moulded in place contained 1 part of cement, 2 parts of sand (of which 1 part was stone-crusher sand), 2 parts gravel (of which 1 part was granolithic) and 3 parts of broken stone. The outer faces of the walls and decks, for a thickness of about 3 ins., were made of granolithic concrete.

A floating plant was used in making the concrete to be moulded in place. During the season of 1899 this plant was carried on a large dump-scow. The mixer, located on one end of the scow, was exactly like the one used for making the blocks. The derrick tending the mixer was on the same end of the scow. The materials were stored in the old pockets, which held enough for two days. The work done by this plant was in every way satisfactory; but, as the scow was not available for the season of 1900, three smaller scows were used. The mixer and two derricks, with the necessary boilers and engines, were placed on one scow, the other two being used as tenders. The decks of the tenders were enclosed, with sides 2 ft. high, and were divided into six compartments for the six different materials, and each scow was designed to carry enough material for half a day's mixing. To prevent the mixer scow from rolling, a guy-rod connected the top of the high derrick mast to the center of the tender scow.

The moulds for the concrete in place were of 3-in., planed and matched pine, braced with 6 x 12-in. timbers, and held together by $\frac{3}{4}$-in. tie-rods. The tie-rods in the lower row were embedded in the concrete during the construction of the wall, and were sawed off flush with the face of the concrete after the moulds were removed.

The usual procedure, in moulding the concrete in place, was to build the lake wall, harbor wall and cross-parapet walls, in the order named. Then the parapet deck was built, which completed a section of the parapet. Sometimes several sections of the parapet were completed before any work was done on the banquette. As stated before, this was left for rough-weather work. The moulding of the banquette deck completed the section, except the placing of the manhole covers. In only two instances was a failure made of completing the walls or deck of a section in one day.

The lake wall of Section 1 was the first concrete to be moulded in place, and nearly two days were required to complete it. The concrete was deposited and tamped in place in a manner similar to that followed in making the blocks. The tops of the decks were rubbed to a smooth surface with a long wooden straight-edge.

An interesting phenomenon occurred during a severe storm in the fall of 1899, when four of the manhole covers were lifted from their seats on the parapet deck and two were washed overboard, while the other two were found on the banquette deck. These manhole covers are circular plates of concrete, 3 ft. in diameter and 6 ins. thick, weighing about 530 lbs. Evidently, the cause of this phenomenon was the high air pressure under the concrete shell, caused by the wave pulsation.

The last concrete was placed on October 25th, and the last work, which consisted of fitting and placing concrete manhole covers, was done on October 28th, 1900.

The cost of this work is given in Table No. 6.

TABLE No. 6.

Removing 1 015 lin. ft. of old superstructure	at	$6.75....	$6 851.25
New hemlock timber, 11.780 ft., B. M.	"	24.00....	282.72
Concrete, 9 294.5 cu. yds.,	"	6.64....	61 715.49
New stone for crib foundation, 222 tons,	"	0.66....	146.52
Timber crib filled and complete in place.....			1 475.00
Manhole covers, 42,	at	$1.25....	52.50
Mooring rings, 19,	"	30.00....	570.00
Total.....			$71 093.48

Extra Work and Materials:

Building stairs, and dressing off concrete.....		$122.50	
Excavating trenches, and fixing manholes.....		114.80	
Capping old piles, and driving new piles in foundation		1 221.87	
Oak piles, 1 550 ft.,	at $0.25.....	387.50	
Lumber, 392 ft., B. M.,	" 35.00	13.72	
Iron bolts and rods, 4 139 lbs.,	" 0.06.....	248.34	
Rip-rap, 70 cu. yds.,	" 2.00.....	140.00	
			2 248.73
Grand total.....			$73 342.21

Length of work, 1 015 ft.; cost per linear foot, $72.26

Extension to Stony Point.

A great increase in the commerce of the harbor of Buffalo, and lack of room in the narrow channels of the inner harbor, namely, Buffalo Creek and the City Ship Canal, had for some years given rise to discussions and plans for increasing the harbor facilities. The Lehigh Valley Railroad Company had constructed at Tifft Farm, the head of the City Ship Canal, an extensive system of slips, docks and warehouses with every facility for carrying on an immense lake business through its individual line of steamers. To reach this port, vessels were compelled to traverse a narrow channel, nearly 3 miles long. A short canal from the lake to the interior canals would obviate this, but its building was precluded by the want of a suitable breakwater to protect it. This breakwater would also enlarge the outer harbor, affording increased anchorage area for vessels,

and form a protection for the lake front southward to Stony Point, so that this locality could be utilized for commercial purposes by piers and docks built out into the harbor thus formed.

The complete wrecking of the shore-arm breakwater, on October 14th and 15th, 1893, tended to increase the public desire for a larger harbor, and, as a result of this agitation, Congress passed an act, under date of August 17th, 1894, which provided for a survey and plan for extending the outer breakwater from a point at or near the present outer breakwater southeast to a point at or near Stony Point.

In accordance with this act, test borings were made, on the line of the proposed extension to Stony Point, which showed that the material overlying the rock was nearly all soft clay, 30 to 40 ft. thick, that the rock was at a depth of from 60 to 70 ft. below the water surface, and that the clay stratum was much too soft to sustain the crib-work-type of breakwater.

Major E. H. Ruffner, the Engineer Officer in Charge, submitted a plan for the extension which provided:

First.—To complete the shore arm as originally proposed. This would be done by extending the sand-catch pier 525 ft., into 18 ft. of water, and reconstructing the shore arm, 2 750 ft. long. This project would secure the harbor area recommended by the Boards of 1868 and 1874.

Second.—To construct a detached breakwater, about 2 000 ft. long, and placed so as to protect a proposed jettied channel connecting the lake with the wharves of the Lehigh Valley Railroad Company. This new entrance channel to be built and maintained by the railroad company.

Third.—To construct a section of breakwater, 1 850 ft. long, extending northward from the Stony Point shore.

Fourth.—When necessity for further protection arose, to fill in the gap between the Stony Point breakwater and that protecting the Lehigh Valley entrance.

The proposed breakwater was to be of stone and concrete, built on a mattress of green trees, with 6 to 12-in. ends, laid in two cross-layers, wired together, and weighted by the usual foundation stone. This mat was to be of a minimum width of 60 ft. for 30 ft. depth. On this mat were to be placed alternate mattresses, of similar character, 30 ft. wide, and 2-ft. layers of sinking and holding stone, until a depth

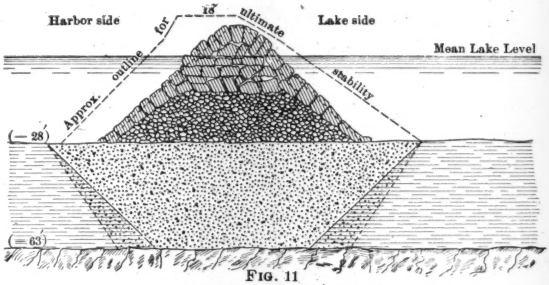


FIG. 11

BOARD OF ENGINEERS' DESIGN.

ESTIMATED TOTAL COST OF 11 500 LIN. FT. AT $191.30⅘ = $2 200 000.

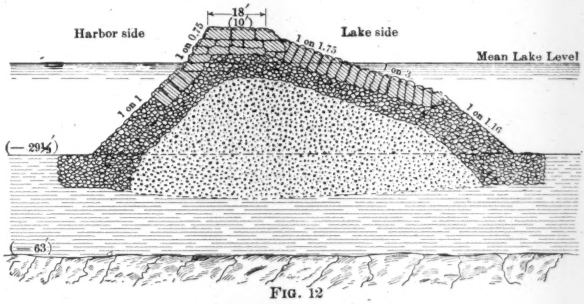


FIG. 12

TYPICAL RUBBLE MOUND DESIGN.

ESTIMATED TOTAL COST OF 12 500 LIN. FT. AT $176.13 = $2 201 650.

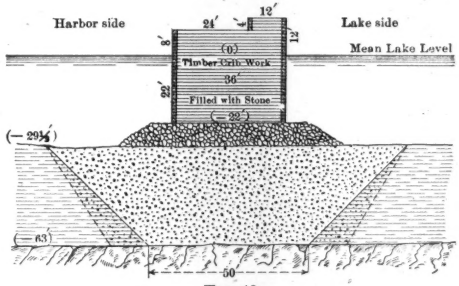


FIG. 13

TIMBER CRIB WORK DESIGN.

ESTIMATED TOTAL COST OF 12 500 LIN. FT. AT $194.00 = $2 424 191.

Scale of Feet

0 5 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150

of 17 ft. was reached. Heavy rip-rap was to be placed over the whole, filling out well the slopes, and raising the structure to 15 ft. depth. Then there was to be a 3-ft. layer of foundation stone, smaller for a bed for the concrete blocks above. The blocks, to begin with, were to be 22 ft. in length, laid crosswise, and reduced in length 2 ft. each until the desired height was reached. These blocks were to be deposited while plastic, and to set while adjusting themselves to what they rested upon.

This plan was not approved, and the whole subject of the further improvement of the harbor was referred to a Board of Engineer Officers for investigation and report.

This Board recommended the partial abolishment of the shore-arm breakwater, and the construction of a breakwater on the line from the existing structure to Stony Point, consisting of a series of rubble mounds, separated by openings about 350 ft. wide. The openings were to prevent the stagnation of the water in the sheltered area and for the convenience of commerce. The breakwater sections were proposed to be five in number, three to be 2 900 ft. long each, one 2 400 ft. long, and a shore arm 400 ft. long, a total of 11 500 ft. They were to be built *en echelon*, to provide the proposed openings.

The construction was to be a mound of small rubble, founded upon a bed of gravel, back-filled into a trench previously excavated to rock, covered with large stone weighing from 4 to 10 tons. The rubble mound was to reach to a height of 12 ft. below mean lake level, and the large stone to a height of 10 ft. above this datum. The estimated cost of the breakwater was: 11 500 lin. ft., at $171 per foot = $1 966 500, or, with contingencies, $2 200 000.

The report of the Board was recommended for approval by the Chief of Engineers, and was approved by the Secretary of War on August 1st, 1895.

The River and Harbor Act, of June 3d, 1896, authorized the construction of the extension of the breakwater to Stony Point, at a cost not to exceed $2 200 000.

Under date of April 6th, 1896, the plans proposed by the Board of Engineers for this extension were reviewed critically by the Engineer Officer in Charge, Major T. W. Symons (see Figs. 11, 12 and 13), and the following modifications were suggested:

1. *Openings.*—That only one opening, 600 ft. wide, near Stony Point,

be required; that this, with the northern main harbor entrance and a fair-weather opening between the existing and proposed work, 250 ft. in clear width, are all the openings required for the convenience of commerce. That the openings proposed by the Board do not curtail the aggregate length of the breakwater, but add expense, as the ends would have to be made much stronger than other portions. The whole structure would be less strong if built in detached portions than if these portions were united into a homogeneous whole; nor are the openings required for the circulation of the water, to prevent it becoming stagnant and offensive. With a large opening, 600 ft. wide, near Stony Point, and the harbor wide open at the north end, there would be a constant circulation of water entering at the south and flowing ever toward the Niagara River. Throughout this narrow eastern end of Lake Erie, there is a constant flow toward the outlet. The natural storm effects would also aid in insuring a full and sufficient circulation of water. The openings are objectionable in permitting waves to enter the harbor and interfere with the anchorage, and also in permitting sand, borne by the general lake currents, to enter the harbor.

2. *The Wrecked Shore Arm.*—That instead of using a portion of this wrecked work, it be entirely removed, as, in its present location and position, it would prove a constant menace to navigation.

3. *Sheltered Area.*—That the Board's breakwater location curtails the sheltered enclosure and protected area. By moving the location of the breakwater farther out, an increase in area of 35% would be obtained.

4. *Length.*—The length of breakwater given by the Board is 11 500 ft. This does not include 1 160 ft. of the wrecked shore arm, requiring reconstruction. This would make a total length of 12 660 ft. The revised location requires a breakwater 12 500 ft. long, with but three exposed ends, as against nine in the plan of the Board.

5. *Depth.*—The depth of water along the line of the proposed breakwater is about 1½ ft. greater than that along the Board's line. The difference in depth to the rock bottom is uncertain, but it is believed that there is practically no difference.

In consideration of the foregoing, Major Symons recommended that the location proposed by the Board of Engineers be abandoned, the same to be placed farther seaward and in slightly deeper water.

The breakwater now proposed would be in two sections, a 2 800-ft. section adjoining Stony Point, and a 9 700-ft. section between the Stony Point section and the existing (old) breakwater. There would be two openings, a main opening, 600 ft. wide, at a distance of 2 800 ft. from Stony Point, and a fair-weather opening, 300 ft. wide, at the junction of the old and new work.

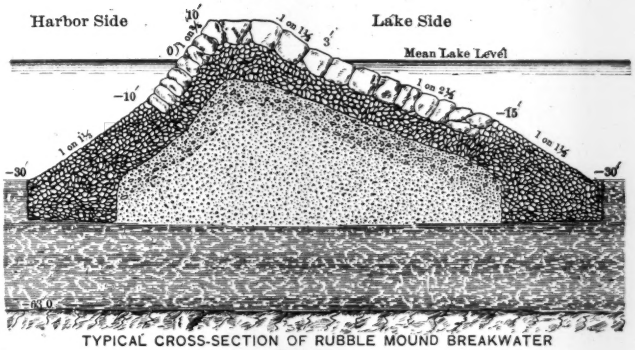
As previously stated, the Board recommended for the whole structure a rubble mound upon a gravel foundation. In place of this, Major Symons recommended a stone structure, with a gravel hearting protected on the sides by rubble, and the whole capped with large stone, weighing 4 tons and upward, carefully placed. This large stone was to come down to a 10-ft. depth of water. The thickness of this giant paving was to be 8 ft. on top and 6 ft. on the slopes. The smaller rubble was to be 9 ft. thick on top and 6 to 12 ft. thick on the slopes. The entire structure was to be founded on the natural bottom of the lake.

This design was afterward slightly modified. It was recommended that 5 000 ft. of breakwater at the northern end be built of this type and the remaining length, to the harbor entrance, 4 700 ft., be built of timber cribwork of the dimensions of the existing breakwater, *viz*, cribs 22 ft. deep and 36 ft. wide, covered with a superstructure 6 ft. high on the harbor side and 12 ft. high on the lake side.

The relative lengths of the stone and timber-crib breakwaters were subsequently changed to 7 250 and 2 739 ft., respectively. The Stony Point section, 2 800 ft. long, was to be of similar cribwork, reduced in size in shoal water. All cribwork was to be founded upon a rubble foundation, about 8 ft. high, resting upon gravel, back-filled into a trench excavated to rock.

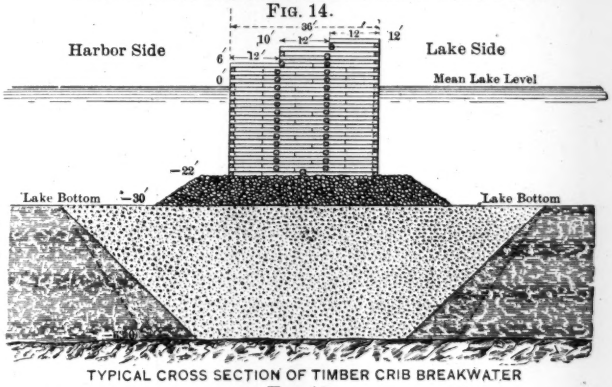
The plans of Major Symons were duly approved by War Department endorsement, of August 11th, 1896, and, in accordance with the authority contained in the River and Harbor Act of June 3d, 1896, the work of extending the breakwater to Stony Point and the sand-catch pier to the pierhead line was advertised, and bids opened on November 30th, 1896. The contract (which is a continuous one) was awarded to the lowest bidders, Messrs. Hughes Brothers and Bangs, of Syracuse, N. Y., on January 27th and 28th, 1897.

Figs. 14, 15, 16 and 17 show the location and types of structures under which proposals were canvassed.



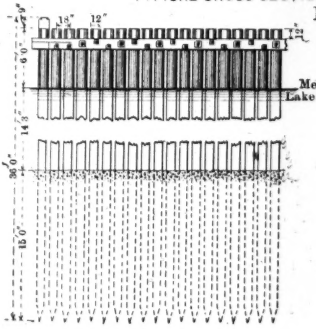
TYPICAL CROSS-SECTION OF RUBBLE MOUND BREAKWATER

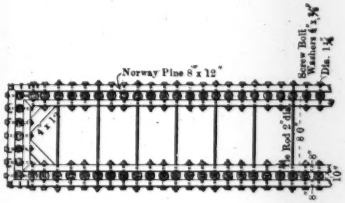
FIG. 14.



TYPICAL CROSS SECTION OF TIMBER CRIB BREAKWATER

FIG. 15.





SAND-CATCH PIER EXTENSION

FIG. 16.

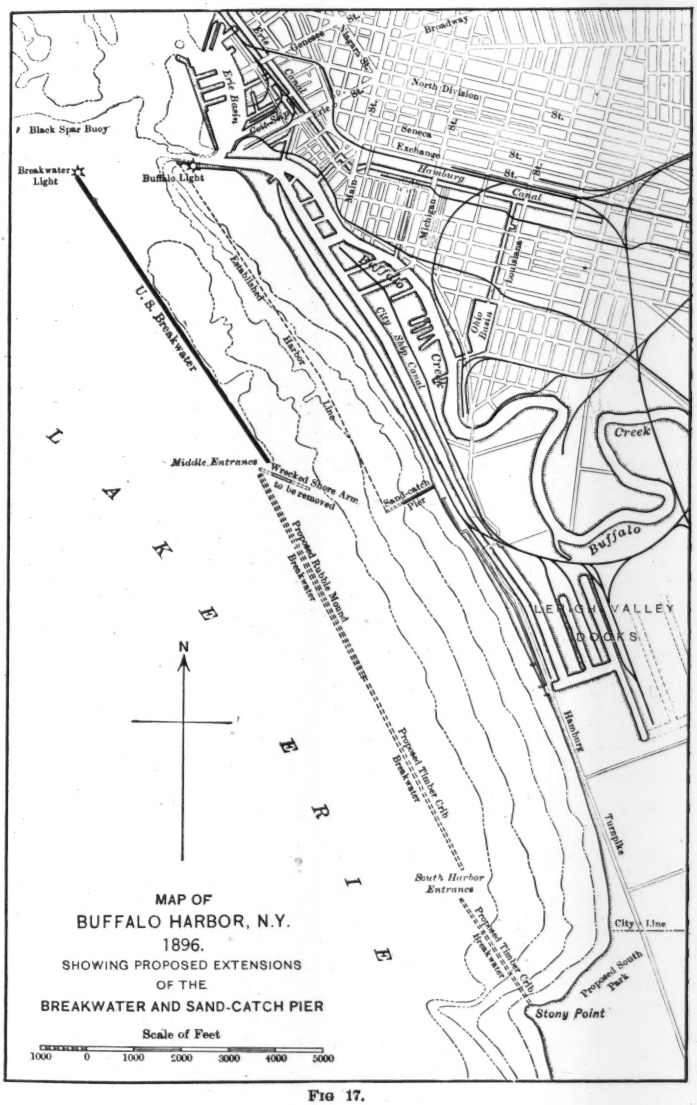
Table No. 7 contains a detailed statement of the quantities, according to the original estimate and the prices bid:

TABLE No. 7.

RUBBLE-MOUND BREAKWATER, 5 000 FT. LONG.					
Gravel foundation and hearting	640 000 cu. yds.,	at	$0.13	= $83 200.00	
Small and ordinary rubble	400 000 tons,	"	0.80	= 320 000.00	
Large capping stone	220 000 "	"	1.25	= 275 000.00	
					$678 200.00
or $135.64 per linear foot.					
TIMBER-CRIB BREAKWATER, 4 700 FT. LONG, FROM RUBBLE-MOUND BREAKWATER TO SOUTH HARBOR ENTRANCE.					
Trench excavation	565 000 cu. yds.,	at	$0.18	= $101 700.00	
Gravel for trench filling	600 000 "	"	0.13	= 78 000.00	
Rubble foundation	116 000 tons,	"	0.80	= 92 800.00	
Timber cribwork	4 700 lin. ft.,	"	92.37	= 434 139.00	
Iron mooring cleats	25,	"	6.00	= 150.00	
Iron mooring posts	25,	"	36.00	= 900.00	
					707 689.00
or $150.57 per linear foot.					
TIMBER-CRIB BREAKWATER, 2 800 FT. LONG, FROM SOUTH HARBOR ENTRANCE TO STONY POINT.					
Trench excavation	240 000 cu. yds.	at	$0.18	= $43 200.00	
Gravel for trench filling	250 000 "	"	0.13	= 32 500.00	
Rubble foundation	44 000 tons,	"	0.80	= 35 200.00	
Timber cribwork	2 800 lin. ft.,	"	92.37	= 258 636.00	
Iron mooring cleats	10,	"	6.00	= 60.00	
Iron mooring posts	10,	"	36.00	= 360.00	
					369 956.00
or $132.12 per linear foot.					
SAND-CATCH PIER EXTENSION.					
Piles	450,	at	$15.00	= $6 750.00	
Waling	1 200 lin. ft.,	"	0.25	= 300.00	
Tie-rods, screw-bolts and straps	15 425 lbs.,	"	0.02½	= 385.63	
Transporting pile shoes	200,	"	0.25	= 50.00	
Stone filling	2 000 cu. yds.,	"	1.06	= 2 120.00	
					9 605.63
Total estimated cost					$1 765 450.63

Soon after being awarded the work, the contractors proceeded to establish the extensive plant required for the proper and energetic prosecution of the work. Having decided to procure large quantities of stone from Canada, they purchased a tract of land, about 80 acres, at Windmill Point, Ontario, on which a quarry was opened. To reach the navigable waters of Lake Erie, a narrow-gauge railway, about 1 mile long, was necessary, and also the construction of a loading dock, from which to transfer stone to barges.

The contractors also arranged to procure rubble from the extensive spoil banks on the Erie Canal, about 1 to 2 miles west of Lockport and



MAP OF
BUFFALO HARBOR, N.Y.
1896.
SHOWING PROPOSED EXTENSIONS
OF THE
BREAKWATER AND SAND-CATCH PIER

Scale of Feet
1000 0 1000 2000 3000 4000 5000

FIG 17.

30 miles east of Buffalo. These spoil banks consist of limestone, excavated from the prism of the Erie Canal during its construction, and deposited on either side of the same. Suitable machinery to transfer this stone to canal-boats was installed there.

For the purpose of building timber cribs, a tract of land bordering on one of the slips owned by the Lehigh Valley Railroad Company, at the southern end of the City Ship Canal, was procured. This site also served as a storing yard for the immense quantities of pine and hemlock needed in the work. A building containing framing and gaining machines, circular saws, boiler, engine and air-compressor plant was also erected there. Docks and ways were also constructed.

They assembled a fleet of canal-boats, deck and dump scows, and tugs, and made arrangements for the construction of a large clam-shell dredge.

The first work on the breakwater extension was to determine the axes of the two main sections. The axis of the stone breakwater and South Harbor section passed through a point 250 ft. seaward and at right angles to the same. This point was obtained by measuring the distance with a line and sighting across the free end of the old breakwater, which presented a plane surface, 36 ft. long. A pile was driven, and the point thus marked was designated as Station 0, being the initial point or beginning of the stone breakwater. It may be stated here that this pile was driven through Crib No. 1 of the wrecked shore-arm breakwater described previously.

The axis of this breakwater also passed through the old Lake Survey triangulation station at the extremity of Stony Point, which was marked by an iron bolt, about 2 ft. long, driven into the sand, with the top about 1 ft. below the surface. A target, consisting of a timber tripod, was placed over the bolt. An auxiliary target was also placed on the old breakwater on the axis of the new work. The two targets are nearly 16 000 ft. apart.

The axis of the Stony Point breakwater was also fixed, by large wooden monuments, placed close to the surface of the ground, and by a range target, some 20 ft. in height, to be used during dredging operations.

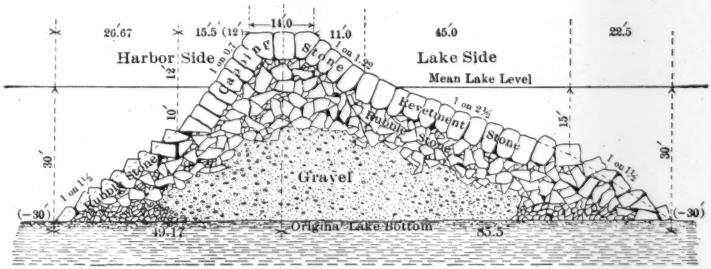
The axis of the stone breakwater lies in the middle of the completed top, that of the South Harbor breakwater lies in the middle of the parapet, or 6 ft. from the lake face, while that of the Stony Point

breakwater is the lake face. These are the axes used for construction purposes. The two main axes cross each other at a point about 777 ft. from the south end of the Stony Point breakwater, measured along the lake face of the same. The angle of intersection is 8° 53′. The bearing of the Stony Point breakwater was determined as S. 35° 57′ E., making the direction of the stone breakwater, also that of the South Harbor section, S. 27° 04′ E.

The construction of the extension to Stony Point was begun by the contractors, on May 19th, 1897, by placing rubble for the stone breakwater, on the lake and harbor sides of the footing, of the prescribed cross-section.

The Rubble-Mound or Stone Breakwater.

The essential features of this breakwater are, the rubble ridges at the foot of the slopes on the lake and harbor sides, the gravel hearting, the rubble facing confining this hearting, and the superstructure of large revetment and capping stones.



CROSS-SECTION OF STONE BREAKWATER
EXTENSION TO STONY POINT
Scale of Feet
0 10 20 30 50 60 70 80

Fig. 18.

Fig. 18 is a cross-section of this breakwater as finally modified. The dimensions in 30 ft. of water, the prevailing depth, are as follows: Width at the bottom, 134.67 ft.; at the water line, 40.95 ft., and on top, 14.0 ft. The lower harbor slope is 1 on 1½, to a point 10 ft. below water level, at which point the harbor revetment* of large stone blocks

*These blocks, when under water, are termed revetment stone, and, when above water, capping stone.

begins; then the slope changes to 1 on 0.7045. The lower lake-side slope is 1 on 1½ to a point 15 ft. below lake level, which is the starting point of the lake revetment; the slope of the revetment is 1 on 2½ to a point 3 ft. above water level, and above this the slope is 1 on 1.22. The height above water contemplated originally was only 10 ft., with a width of 15 ft. on top; but the height was changed to 12 ft., and the width reduced to 14 ft., hence the decimal expression of the slopes.

The theoretical cross-sectional area of the structure is 2 996.42 sq. ft., divided roughly as follows: Gravel, 950 sq. ft.; rubble, 1 450 sq. ft., and revetment and capping stone, 600 sq. ft. In actual construction, the area of the rubble was somewhat increased, as the slopes of both the lake and harbor sides were built somewhat beyond the prescribed lines.

Actual work on the structure was commenced on May 19th, 1897, and was practically completed on December 10th, 1902, on which date, and a few days later, 1 300 ft. of the harbor side of the structure, between Stations 59 and 72, sank about 12 ft. Reference to this damage will be made later. No work was done during the winter months.

Methods of Construction.—The position of the breakwater having been fixed by targets on the prolongation of the axis, piles were driven on the axial line, at 1 000-ft. intervals from the north end, and numbered 0, 10, 20, 30, etc., thus dividing the work into 1 000-ft. sections. Piles were next driven at 250-ft. intervals, at the foot of the prescribed slopes of the lake and harbor sides. The latter served as guides in placing the materials. The outside ridges were built first by dropping small limestone rubble over the sides of the canal-boats, in which the material was first brought to the work. The loaded boats were towed to Buffalo by a small steam tug. On arrival at the breakwater site, they were carefully aligned, parallel to the axis of the work, and with one side over the proposed lake- or harbor- side ridges. Then the stones were dropped one by one. These ridges were built about 20 ft. wide, and from 6 to 8 ft. high.

The usual load for a canal-boat was 200 cu. yds., or 225 tons. Usually, three boats were unloaded daily, with a force of thirty-five men. A fleet of nine canal-boats was in service, of which three were loading at the quarry, three were on the way, and three were being unloaded.

FIG. 1.—STONE BREAKWATER. NORTH END. LOOKING TOWARD STONY POINT. 1899.

FIG. 2.—STONE BREAKWATER. HARBOR SIDE. STONE BLOCKS WEIGHING 17 TONS. 1902.

Before unloading a boat its longitudinal position was determined carefully by measurements taken from the nearest pile; its transverse position or range was taken at the same time. These notes, together with the name of the boat, the number of the ticket accompanying it, the date, time of unloading, and other pertinent information, were recorded in blankbooks prepared for the work. While unloading, the boats were held in position by lines running to anchor buoys.

After a sufficient length of these rubble ridges had been constructed, the gravel hearting was begun by depositing the gravel from dump scows.

The gravel was dredged from the bed of the Niagara River, between the foot of Ferry Street, Buffalo, and the International Bridge. Owing to the swift current of the river, especially at the water-works intake crib, where it is more than 7 miles an hour, there was great difficulty in towing the loaded scows up stream. After repeated trials with the large harbor tugs of Buffalo and Detroit River as motive power, a large and powerful tug was built especially for that service.

The gravel was usually brought to the work in dump scows, also built for the work. There were four of these scows, two of wood and two of steel, each having eight pockets. The capacity of each scow was 590 to 616 cu. yds., but, generally, they carried only from 450 to 470 cu. yds., one pocket being left empty to facilitate towing.

After a sufficient quantity of gravel had been deposited, generally a layer of 2 to 3 ft. in thickness, the rubble facing was commenced. The facing was of large and small rubble from the contractors' Canadian quarry, about 6 miles from Buffalo, and was brought to the work in dump scows.

Soon after work was commenced, it became apparent that some positive and reliable cross-ranges were necessary, to regulate the method of dumping materials. This was accomplished by stretching, between the piles at the 1 000-ft. stations, wire cables, on which wooden markers were suspended at fixed intervals. One end of the cable was attached firmly to a pile, and the other end was passed through a pulley on the opposite pile, and weighted to keep it taut. This prevented the breaking of the cables when the piles were swayed by the waves.

The position of every scowload of gravel, or rubble, was recorded in the same manner as the canal-boat stone. These data were plotted

on cross-section paper, each scow being shown in plan. With the large number of scow loads of material of different kinds, it soon became evident that this method was quite inefficient, and it was decided to cross-section the deposits regularly. A 20 x 50-ft. scow, drawing about 1 ft. of water, was utilized for this purpose. The cross-sections were taken by soundings, the scow being placed normal to the axis and held in position by anchor buoys. The soundings were taken at stations 10 ft. apart, measured along the axis of the breakwater, and at 5-ft. intervals at right angles to the same. Small flexible wire cords were used in measuring. The larger were $\frac{1}{8}$ in. and the smaller $\frac{1}{16}$ in. in diameter. The larger cords were graduated every 20 ft., and the smaller ones every 5 ft. The graduations were marked by small loops of wire passed around the cords. One end of the wire was wound around the cord several times, then a piece of $\frac{1}{2}$-in. round iron was held against the cord, and the wire passed around both several times. It was then cut off, and the bolt withdrawn. In the case of the large wire, the resulting loops served to attach wooden floats held in place by harness snaps. The 20-ft. floats were painted white and the 100-ft. floats red. The heavy wire was usually 1 000 ft. long, and the floats kept it from sinking. On the smaller wire numbered brass tags indicated the distances.

The soundings were plotted in $4\frac{1}{2}$ x $7\frac{1}{2}$-in. cross-section books, to a scale of 20 ft.=1 in. To facilitate the work, a skeleton cross-section was plotted previously on each page, a separate book being used for each 1 000-ft. section.

These cross-sections served the purpose admirably, and gave a graphic record, which showed at a glance the state of the work under water.

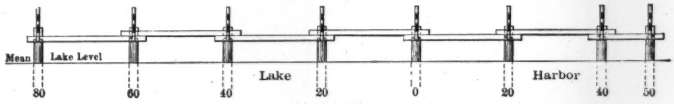
Owing to the short length of the sounding scow, which required a number of settings to cover the full width of the work, a larger and more efficient one was built. In order to secure as many cross-sections as possible without changing the scow's position, it was built with two hulls, each 10 ft. wide and 120 ft. long, separated by an open space, 10 ft. wide, thus forming a catamaran. By this arrangement four parallel cross-sections, 10 ft. apart, could be sounded from one position of the scow.

Two 8 x 10-ft. deck houses were built on the scow, one to serve as a plotting and computing room and one as a store-room for lines, etc. There were crank capstans at each end for handling anchor and moor-

ing lines. The hulls were only 3 ft. in depth, in order that the deck might be as low as possible to facilitate sounding operations, and were stiffened with suitable trussing and bracing. The vessel was most appropriately named the *Gemini*.

An inspection of the prescribed cross-section, Fig. 18, will show what an important factor this sounding was. Thousands of tons of material were being deposited daily, actually out of sight, its position being known only approximately at the moment of dumping. From an economical point of view, it was very important to know that no material was wasted outside of the prescribed slopes, and that the proper co-relation between the gravel and rubble was being maintained. This information could only be obtained by careful soundings, taken unremittingly, and following closely the depositing of material at all times. To complicate matters further, berms had to be left for the placing of the initial revetment stone on both the lake and harbor sides, at 15 and 10 ft. below water level, respectively. This required very careful dumping, and, of course, satisfactory results were only obtained by skilfully placing the dump scows on the proper ranges.

The original suspended cable ranges not proving entirely satisfactory, so-called fences, Fig. 19, were erected at each 1 000-ft. station.



ALIGNMENT FENCE.
FIG. 19.

They were composed of a row of piles, driven at 20-ft. centers, to the sides of which, near the tops, heavy planks were bolted, and to which the range sights were attached. The latter were light strips of wood, 2 to 3 ins. wide and from 3 to 6 ft. long, painted white and red, so as to be readily discernible.

The gravel was deposited to a height of 10 ft. below mean lake level, and then covered with rubble. This height was controlled, to some extent, by the draft of the loaded scows, and therefore much of the rubble over the gravel hearting had to be unloaded by floating derricks, instead of by dumping.

After the gravel had been thoroughly covered, the mound was ready to receive the revetment stone. Owing to the large size of the

rubble used for the gravel facing, its surface was necessarily uneven and not in proper condition to receive the revetment blocks. Therefore it had to be prepared properly before any of these blocks could be placed. On the lake side this was done by a method termed leveling, which consisted in dropping small stone by hand over the side of a scow having one side aligned exactly parallel to the axis of the work. Blue-print tables had been prepared showing the proper depth of the prescribed slope below mean lake level for each 2-ft. horizontal interval. The leveler in charge of the work had a copy of these tables, and, by placing the deck scow on the proper range, the desired depth was known at once. While the stone was being thrown over the edge the leveler passed back and forth constantly with a sounding pole, regulating the deposit. The scow was always worked in from the outside toward the center, moving 2 ft. at a time. By this means a fairly uniform and smooth slope was secured.

On the harbor side, the foundation for the revetment stone being horizontal, this leveling was more readily done.

The next operation was the placing of the revetment stone. On the lake side, the initial course, which is 63 ft. from the axis, was placed level, for most of the work, although at first the stones were set at an angle. The succeeding stones were then placed so as to be normal to the slope. This was attained readily by hanging them at the corners.

On the harbor side, the initial stone was triangular in shape, and was set with the apex up. The position of this apex was determined by ranges. See Fig. 20.

The revetment stones were placed without a diver, and with sounding poles in the beginning. At times the water was sufficiently clear and the light favorable for seeing the stone with the unaided eye, down to a depth of about 10 ft., under which conditions the work was much simplified.

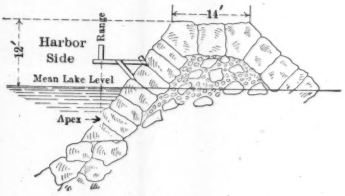
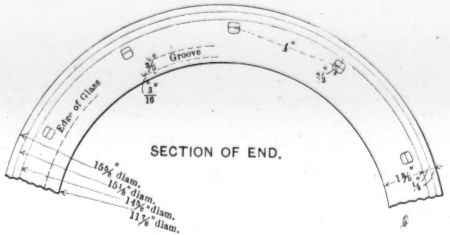


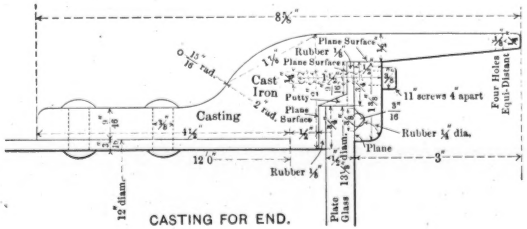
FIG. 20.

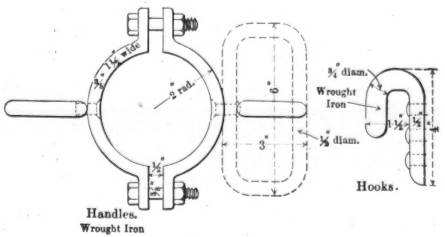
These favorable conditions of light, however, were not sufficient for the necessary examinations and inspections of the placement of the revetment stone, and therefore, water telescopes were constructed to aid in such inspection.

WATER TELESCOPE.

Estimated Displacement, 223.0 lbs.
" Weight, 261.5 lbs.







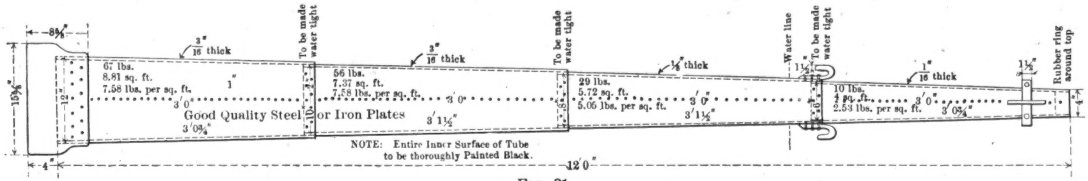


FIG. 21.

The tubes of the larger telescopes are of $\frac{3}{16}$, $\frac{1}{8}$ and $\frac{1}{16}$-in. steel plates, riveted together and made water-tight. The tubes are 12 ft. 4 ins. long, with diameters of 12 ins. at the object end and 4 ins. at the eye end. They are fitted with handles, and with hooks for tackle to raise and lower them, and are painted black inside. A casting, riveted to the object end of the tube, is provided with planed seats for a plane object glass, 13 ins. in diameter and $\frac{1}{2}$ in. thick, and for the rubber packing and ring necessary to make water-tight joints.

Shorter telescopes were made afterward, to suit shallower water, and there were three lengths in use, namely, 12 ft. 4 ins.; 9 ft. 10 ins., and 7 ft. 4 ins., weighing, respectively, 301, 222 and 171 lbs. These weights are somewhat less than the displacement of the tubes, thus allowing them to float about 2 ft. out of water. See Fig. 21.

The revetment blocks, which are quarried out approximately cubical in shape, with one dimension as near 6 ft. as possible, were lowered into place in juxtaposition, as nearly as practicable, so as to cover fully the rubble behind them. Of course, their approximation to the prescribed slope under water is governed entirely by the prepared foundation upon which they rest, yet it is proper to state that the variation from the prescribed lines is quite small, as frequent soundings on the finished work have demonstrated.

As the revetment stone approached the water surface, any little inequalities in line and surface were rectified, in order to obtain as straight a "shore line" as possible. At this stage the raising of the mound was followed by the deposit of more rubble, both large and small, conforming as closely as possible to the prescribed cross-section. The capping stones were then set as compactly as possible, with due regard for close joints and beds, and were well bedded.

Templates giving the prescribed cross-section, placed at close intervals, aided in placing the capping stone on the required grade.

The extreme top of the structure is composed of two trapezoidal-shaped stones and a central filler. These odd-shaped stones were especially quarried by drilling and blasting, no cutting tools being used.

All the revetment and capping stones were placed by floating derricks, moored alongside the work, and held in position by hawsers leading to anchor buoys. The scows loaded with stone were placed alongside the derrick on the outside. When engaged in setting revetment stone on the lake side, small floats, averaging about 14 x 20 ft. in size, were placed between the floating derrick and the work. These floats served

as a base for the workmen manipulating the swinging stone, and also for the Government inspector.

A working crew consisted of eight men, as follows: One foreman, having general oversight and selecting the proper stone to set; one assistant foreman, giving directions about swinging the stone, and raising and lowering the boom; two engine men, one of whom operated the main hoisting engine, the other the swinging engine; two stone pickers, picking holes for the stone hooks, and two laborers on the wall setting the stone.

The setting of the revetment and capping stone on the lake side was oftentimes retarded and suspended during westerly winds, when no work could be done on that side. This delay, later, was diminished to some extent by substituting for the original short booms of two of the derricks, two longer ones, of 75 and 90 ft., thus making it possible to place stone from the harbor side during stormy weather.

At the beginning of the season of 1899, the manner of laying the massive capping stone on the lake side was changed. During the previous season this capping stone had been laid with the longer axes of the stones perpendicular to the axis of the structure, breaking joints as well as practicable. In doing this there was some difficulty in getting close joints, which were the more objectionable as they ran from a point 5 ft. below mean lake level to the top of the rubble-mound structure. It was then decided to lay the stone parallel with the axis of the structure, the same as the revetment. This change produced a better class of work by making closer horizontal as well as perpendicular joints.

The several kinds of material, small rubble, large rubble, gravel and large revetment and capping stone, were conveyed to the work in vessels. A part of the small rubble, from quarries on the Erie Canal, was conveyed in canal-boats. The remainder of the rubble, both small and large, was conveyed by dump and deck scows, the capping only on deck scows and the gravel only in dump scows.

When bids for the work were invited, originally, prices for the rubble were asked, both by measurement by the cubic yard, and by displacement by the ton. The latter method, however, was adopted for payment, as more accurate results were secured.

As a matter of information, however, the measurement of all stone brought from Lockport, during the season of 1897, was determined by both methods, and, from a large number of canal-boat loads, it was

found that 1 cu. yd. equalled 1.125 tons. This was limestone, and averaged $166\frac{2}{3}$ lbs. per cubic foot. It will be observed that, as loaded in the canal-boats, the voids were exactly 50 per cent. All vessels carrying material were measured accurately, and the displacement of canal-boats and deck scows was computed carefully from the data thus obtained. For the dump scows only the cubic contents of the pockets were determined.

All the vessels, except the gravel scows, were provided with glass water-gauges, located amidships, in the bow and the stern. Each gauge consists of a 3-in. wrought-iron stand-pipe screwed into the bottom of the boat. A glass tube, similar to those used for steam boilers, is connected with the stand-pipe by brass fittings. A hard-wood rule, graduated to feet, tenths and hundredths, is placed alongside the glass tube, for recording the height of the water. The rule is so placed with reference to the bottom of the boat that the readings give at the same time the exact draft of the boat. The capacity of the scows, loaded to various depths, was readily determined from displacement tables carefully prepared for each scow.

To check the calculated displacement of the scows, test weighings were made at the opening of each season. These consisted in weighing one or more loads on a track scale and making comparison with the tabulated results. A surprising agreement was found in nearly every case, the difference being only trifling.

Large quantities of rubble were brought to the work in dump scows. As the loads could not be determined by displacement, the carrying capacity of each particular scow was ascertained by reading the gauges when light, and at frequent intervals during loading, the stone all the while being weighed on a reliable track scale. By simply dividing the difference of the gauge readings into the weighed loads a ready multiplier for future loads was obtained. It may be noted here that this multiplier varied somewhat with the size of the stone.

The gravel was measured in the pockets of the dump scows. Suitable tables, giving the contents of the pockets to the deck level, and for fractional parts of feet above and below this datum, were prepared, and from these the contents of full or partial loads could be ascertained quickly.

Each scow was numbered or named, and it and its load were identified by a "loading ticket," recording the light and loaded gauge

readings, the quantity and kind of material composing its load, its running number, and the date. These tickets were issued and signed by the inspector at the place of loading. The ticket accompanied the loaded scow to the work, and a check record of the loaded and light readings, of the kind and quantity of material, and when and where placed, was marked thereon. These tickets afterward formed the basis for the reports and records of material for the monthly payments.

The construction of the stone breakwater occupied six seasons, a season's work being generally seven months, or from May to November, inclusive.

The contract length of the stone breakwater was 5 000 ft., but, while the work was in progress, Major Symons recommended that the timber cribwork section be reduced, thus adding 2 250 ft. to the stone breakwater, making its length 7 250 ft.

The gravel hearting was only delivered in large dump scows, towed up the Niagara River, during the years 1897, 1898 and 1900, practically none being placed in 1899. That deposited during 1901 and 1902 was delivered in small dump scows, towed up the Black Rock Harbor and Erie Canal, entrance to the latter being obtained through the so-called ship lock, connecting with the Niagara River immediately below the International Bridge.

In addition to the rubble from the United States and Canadian quarries, a considerable quantity was dredged from the wrecked shore arm and obtained from blasting operations in Buffalo Creek.

Table No. 8 shows the quantities of material deposited in the stone breakwater (rubble mound) for each year under construction, also the cost:

TABLE No. 8.

Season of:	Gravel, in Cubic Yards.	Rubble.		Capping stone, in tons.	Total cost.
		From United States, in tons.	From Canada, in tons.		
1897	73 184	76 417	27 501	383	$93 127.98
1898	49 569	38 802	66 681	16 079	110 928.84
1899	942	3 932	26 351	42 425	74 562.85
1900	23 781	37 292	79 685	54 989	158 675.09
1901	70 950	27 444	145 814	77 532	242 120.88
1902	26 816	19 504	118 376	93 471	228 724.12
Totals	245 242	203 391	464 408	284 879	$908 139.76

The work was practically completed on December 10th, 1902, on which date 1 000 ft. of the harbor side settled, followed on December 14th, 1902, by 300 ft. more. At this time the detailed cost was as given in Table Nos. 9 and 10.

TABLE No. 9.—Detailed Cost of Stone Breakwater.

Gravel	245 242	cu. yds., at	$0.13	$31 881.46
Rubble, from United States	202 969.3	tons, "	0.80	162 375.44
" " "	422.1	" "	0.77½	327.13
" " Canada	317 606.9	" "	0.80	254 085.52
" " "	16 315.5	" "	0.77½	12 644.51
" " "	129 921.9	" "	0.75	97 441.43
" " "	563.5	" "	0.72½	408.54
Capping stone, from Canada	142 429.1	" "	1.25	178 036.38
" " "	142 449.4	" "	1.20	170 959.28
Total cost				$908 139.69

The difference between the totals of Tables Nos. 8 and 9 is due to the fractional half-cents in the monthly payments.

TABLE No. 10.—Quantities of Materials in Stone Breakwater, and Cost.

Kind of Material.	Quantities.		Cost.	
	Total.	Per linear foot.	Total.	Per linear foot.
Gravel	245 242 cu. yds.	34	$31 881.46	$4.40
Rubble	667 799.2 tons.	92	527 282.57	72.73
Capping Stone	284 878.5 tons.	39	348 975.66	48.13
Totals			$908 139.69	$125.26

Table No. 11 shows the quantities of rubble and capping stone placed in the stone breakwater during the seasons of 1899, 1900, 1901 and 1902, also the estimated amount of labor and the resulting cost per ton of material placed.

Table No. 12 shows the quantities of revetment and capping stone placed in the stone breakwater during the working seasons of 1900, 1901 and 1902.

Administration and Regulations.—Operations on all parts of the breakwater were under the immediate supervision of an assistant engineer, who regulated the contractors' operations so as to secure the specified results, and who had general charge of the inspection and records of the work and was responsible for the official reports.

TABLE No. 11.—Quantities of Rubble and Capping Stone Placed in the Stone Breakwater During the Years 1899 to 1902; also the Estimated Cost of Labor and the Cost per Ton.

Year.	Rubble, in tons.	Capping stone, in tons.	Total tons.	Estimated amount paid for labor.	Cost per ton, in cents.
1899*.....	30 282.9	42 425.1	72 708.0	$13 615.08	19
1900†.....	116 976.7	54 988.7	171 965.4	24 428.06	14
1901‡.....	173 258.2	77 531.7	250 789.9	29 904 95	12
1902‖.....	137 879.8	93 470.9	231 350.7	30 643.76	13
Totals	458 397.6	268 416.4	726 814.0	$98 591.80	13⅓

* To handle the material in 1899 there were generally four floating derricks, one being engaged for part of the time on the adjoining timber cribwork. The average working force consisted of one superintendent, one assistant superintendent, one timekeeper, five foremen, eight enginemen, thirty one laborers, one diver, one diver's tender and six boatmen, assisted by two or three tugs and three yawl boats. Only two scow-loads of gravel, 942 cu. yds., were deposited during the season.

† To handle the material in 1900 there were generally five floating derricks, one of them being engaged for part of the time on the adjoining timber cribwork.

‡ To handle the material in 1901 there were generally five floating derricks. There were, also, six tugs, two as tenders to the derricks, also placing stone and gravel scows, one towing gravel from the Niagara River, one towing rubble and capping stone from the Canadian quarry, one was used as a general utility boat for carrying men, supplies, etc., and one was engaged in watching at night. The quantity of gravel deposited was 70 950 cu. yds. This gravel was towed up the Black Rock Harbor and Erie Canal in small dump scows having an average capacity of 154 cu. yds.

‖ To handle the material in 1902 there were generally five floating derricks. There were also five tugs, exclusive of the tug towing gravel. The gravel hearting was completed on July 1st, 1902, 26 816 cu. yds. having been deposited during the season. The average working force was one superintendent, one timekeeper, five foremen, five assistant foremen, ten enginemen, twenty derrickmen, twenty laborers, one diver and tender, four carpenters, one leveler, one blacksmith, nine yawlmen, one cable splicer, two watchmen and two scowmen.

TABLE No. 12.— Quantities of Revetment and Capping Stone Placed in the Stone Breakwater During the Years 1900, 1901 and 1902.

Year.	Number of working days.	Revetment and Capping Stone Placed.				Average weight of stone, in tons.	Maximum weight of stone, in tons.
		Tons.		Number.			
		Total.	Daily.	Total.	Daily.		
1900.....	164	54 988.7	335.3	8 883	54	6.20	16.5
1901.....	196	77 531.7	395.6	11 729	60	6.60	16.8
1902.....	212	93 470.9	440.9	15 452	73	6.05	18.0
Totals and averages...	572	225 991.3	1 171.8	36 064	63	6.26	18.0

On the rubble-mound construction the placement and measurement of materials were inspected and determined by a chief inspector, assisted by several inspectors as required. The placement of the

under-water materials was particularly important, and required constant supervision. This was especially so in the case of the revetment stones, great care being taken that all stones were in actual contact and set to the required grade. The harbor angle stones required more than ordinary attention. Usually, one inspector was assigned to each of the floating derricks by which all the revetment and capping stones, were set. In addition to their regular duties, these inspectors were required to submit a daily report of operations, giving in detail the kind and number of stone blocks placed, and their position in the structure. The quantity of rubble used, and where placed, were also reported when handled by the derricks. The contractors' force and plant were also noted, as well as the state of the weather, wind and sea. Delays and important happenings were reported as they occurred. Similar reports, on other blank forms, were rendered daily for rubble and gravel deposited.

These daily reports were very useful in keeping the office fully informed as to the state and progress of the work.

Weekly reports of all operations were also submitted, and these formed the basis for the monthly reports and the vouchers for payments.

The trench operations were inspected by an inspector quartered on the dredge. One inspector supervised the timber-crib breakwater construction, including the stone foundation on the trench filling, and another inspector the construction of the timber cribs at the contractors' yard. These inspectors submitted only weekly reports, but daily reports would have been very desirable.

An inspector was also stationed at the contractors' Canadian quarry, and inspected and measured the several kinds of stone. This inspector submitted daily reports giving the details of rubble and capping stone shipped, numbers or names of vessels and their loads in tons. In addition, the contractors' force and plant was reported, as well as the weather, delays and other important items. He also sent in weekly reports.

The sounding operations were under the supervision of an inspector, who directed the work and recorded and plotted the soundings as fast as made. A leadsman made the soundings, and several laborers from the contractors' force assisted in placing the scow in position.

BUFFALO HARBOR

CROSS-SECTIONS OF STONE BREAKWATER

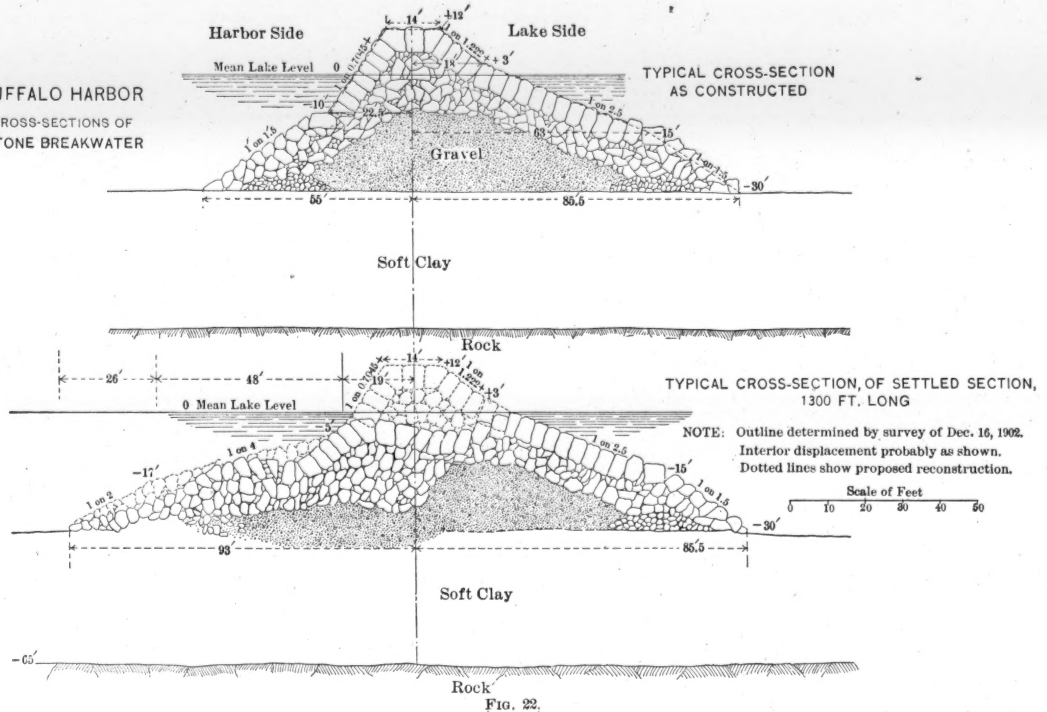


FIG. 22.

Settlement of a Portion of the Rubble-Mound or Stone Breakwater.—The settlement of a portion of the stone breakwater during a severe storm on December 10th, 1902, which was followed by a minor settlement a few days later, has been referred to. In relation thereto, the following, taken from the official report of Major Symons, is of interest. See Fig. 22.

In making the original estimates for this work it was expected that the whole structure would settle into the underlying mud to a considerable depth, and the quantities in the estimate provided for a settlement of 10 to 15 ft.; 10 ft. on the edges and 15 ft. in the center of the structure.

As the work proceeded it was found that the expected settlement did not take place, the total settlement at any place being not more than 1 or 2 ft. The whole rubble-mound breakwater, 7 250 ft. long from its northern end to the junction with the timber-crib breakwater to the south was almost completed, with practically no settlement taking place.

About three days' more work were required to put in place the top course of capping stone at the extreme southern end, and chink a few holes in the capping. The materials necessary for this work were afloat at the site, and the contractors were making the most strenuous efforts to complete the work by working in the inclement season of the year, giving bonuses to their men, etc., when, on December 10th,* 1902, during a violent storm, a length of about 1 000 ft. sank into the clay about 10 to 14 ft., thus postponing the full completion until the next season. Five days later, 300 ft. more, to the south, went down.

The drop commences at 5 900 ft. from the north end of the breakwater and extends to about 7 200 ft., almost to the timber crib. The main portion standing has been completed from one to five years, and, therefore, has had such tests as indicate its permanent stability. The portion that went down was built during the past season. The inference is that the clay bottom at the settled portion has less bearing power than it has to the north, where the structure has stood all right. This inference is strengthened by the fact that the borings taken before the work was started show that in the mud beneath the standing part there were strata of "sand," "hard sand," "gravel," "hard gravel," etc. These strata are entirely absent in the bottom beneath the settled part of the breakwater, where the borings show "soft red clay" and "very soft red clay" only.

*See discussion, pp. 210 and 211.

FIG. 1.—STONE BREAKWATER. LOOKING NORTH, AFTER SETTLEMENT, DECEMBER 10TH–15TH, 1902.

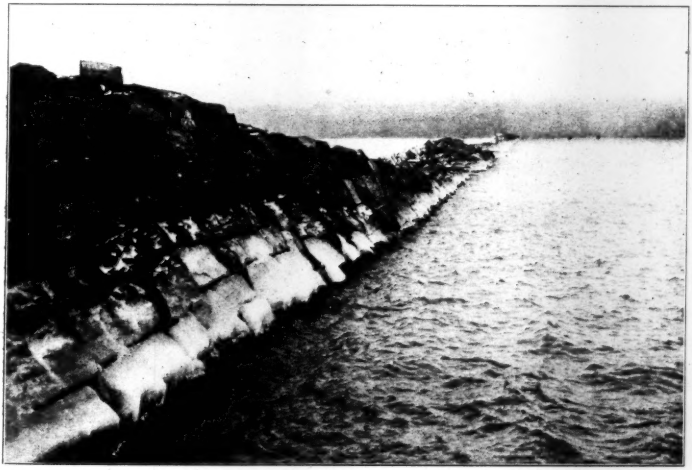


FIG. 2.—STONE BREAKWATER. HARBOR SIDE. STRUCTURE AFTER SETTLEMENT, DECEMBER 10TH–15TH, 1902.

Thus it is clear that the lake bottom with strata of sand and gravel interspersed has sufficient supporting power to sustain the structure without serious settlement, while without these strata it has not, and the breakwater built on it sank to just about the extent which it was estimated the whole would sink at the beginning of the work.

The settlement is peculiar, as only the rear portion sank, leaving the front from the water line lakeward in place and apparently undisturbed. The break occurred at the lake-side water line, the high superstructure and all the landward portion settling, on a very slight angle from the vertical, about to water level, some portions being a little above and other portions a little below water. The movement was apparently a settling and sliding landward under the pounding of the terrific seas. This indicates that the clay has sufficient sustaining power for the front, with its gentle slope and water cushion, but that the steeper rear slope with the battering of the waves was too much for it.

It is believed that the settled portion has reached such a condition as will render it safe to rebuild the superstructure on without fear of further trouble, but, to make the future more assured, it is proposed to correct the irregularities in the harbor slope and widen the base where necessary by placing more stone at the toe of the slope, as generally indicated by Fig. 22, which shows the structure as built, as settled, and as it is proposed to rebuild it.

The proposed method of repairing the damage is as follows:

a.—Place rip-rap along the harbor slope to make the general harbor slope about equal to the lake-side slope.

b.—Remove all capping stone, particularly angle stones, in the sunken portion, which can be conveniently gotten at, and use them again in the new superstructure at the proper time.

c.—Arrange the stone properly as a foundation for a new superstructure.

d.—Build a new superstructure to the same lines as the old one, making the alignment fit in with the standing portions on each side of it.

e.—To add security to the entire structure, it is also proposed to place rubble along the inner slope, thus widening the harbor toe and tending to correct any inclination to settling and sliding.

In performing this work of repair, the original contractors, Hughes Brothers and Bangs, have agreed to take up and reset all the capping stone that can be economically saved, and arrange the settled portion properly as a foundation for the new superstructure, for which work they are to receive their actual force account plus 15% and a fair rental for the derricks, boats, etc., used. All additional materials will be furnished at the original contract prices. It is expected that the entire structure will be completed early in the summer of 1903.

TABLE No. 13.—Rates of Wages Paid on Stone Breakwater.

Superintendent	$150.00 per month.
Diver	125.00 " "
Foremen (and foreman of carpenters)	75.00 " "
Hoist enginemen	65.00 " "
Timekeeper, and cable splicer	60.00 " "
Assistant foremen	55.00 " "
Swing enginemen, watchmen and scowmen	50.00 " "
Derrickmen, pickers and setters	45.00 " "
Carpenters and blacksmiths	2.25 per day.
Diver's tender	2.00 " "
Yawlmen	$1.80 and 2.00 " "
Laborers	0.15 per hour.

Stony Point Section. Timber-Crib Breakwater.

Dredging.—This breakwater extends from Stony Point in a northwesterly direction for 2 802.9 ft., into 30 ft. of water. The entire breakwater, with the exception of a few hundred feet next the shore, stands upon an artificial foundation of sand and gravel, up to the level of the original lake bottom, this material having been backfilled into a trench dredged practically to bed-rock.

The breakwater consists of five sections of timber cribwork of varying dimensions, beginning with the smallest one at the shore end, the depths of the cribs corresponding approximately with the increasing depth of water, until 22 ft. is reached, which is the maximum depth of the cribs. In deeper water they rest on a rubble foundation built up to that level.

TABLE No. 14.—Work Done by Dipper Dredges in the Seasons of 1897 and 1898.

1897.

Name of dredge.	Stations.	Days worked.	Hours worked.	Cubic yards of excavation.	Dates.	Rates.		Material.
						Day.	Hour.	
H. & W., No. 9...	0 to 2 + 40	11	(?)	315*	Aug. 4th to 23d.	28.6	(?)	Sand, hard clay, boulders with gravel.
Brian Boru.....	9 + 35 to 11 + 20	2	17	2 487	June 29th to July 1st.	1 243	146	Some sand; moderately stiff clay.
H. & W. No. 1....	4 + 50 to 3 + 00	6	43¼	1 432	Sept. 3d to 8th.	239	33.1	Clay, sand and boulders.
Brian Boru.....	5 + 00 to 3 + 80	6	54	3 209	Sept. 9th to 20th.	535	59	Hard clay and boulders.
H. & W. No. 1....	7 + 00 to 5 + 40	3½	21	674	Sept. 28th to Oct. 4th.	192	32	Hard clay and boulders.
Total, in 1897, 8 117 cu. yds. = 62 scowloads.								

1898.

Brian Boru.....	5 + 30 to 6 + 10	5	29¾	1 908	March 25th to April 5th.	400	67	Some clay; hard clay and boulders.
Tipperary Boy...	6 + 57 to 7 + 07	1	4¼	325	April 11th.	325	76	Hard clay and boulders; hardpan.
H. & W. No. 4....	5 + 40 to 8 + 05	7	57¾	2 611	April 30th to May 13th.	373	45	Hard clay and boulders; hardpan.
Total, in 1898, 4 934 cu. yds. = 41 scowloads.								

* This was allowed for by computing the required place measurement of the material to be removed to make the foundation.

The first dredging was done on June 29th and July 1st, 1897, in about 15 ft. of water, but, as no material for back-filling was on hand, work was stopped temporarily. Owing to the shallow cutting required at the shore end, the contractors attempted to remove the material by plowing and scraping, but this was ineffectual and had to be abandoned. Then a small dipper dredge of shallow draft (2½ ft.) began work on August 4th, 1897, and dredged out the trench to the depth required for the first section of cribs, 300 ft. long and 4 ft. deep. This dredging was necessarily slow and tedious, on account of the many large boulders in the clay. In deeper water two dipper-dredges of greater draft and capacity worked until October 4th, 1897, when dredging was suspended for the season.

Dredging was resumed on March 25th, 1898, by the dredge *Brian Boru*. During the night of April 5th and 6th, 1898, while being towed to shelter during a heavy storm, the dredge sank, and three men were drowned. Two other dipper dredges completed that part of the work.

The work accomplished by these dipper dredges during the seasons of 1897 and 1898 is shown in Table No. 14.

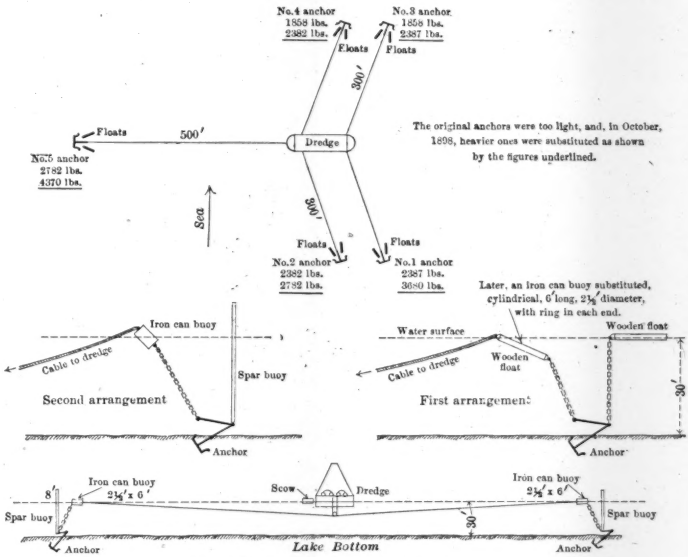
The remainder of the trench was excavated by the clam-shell dredge *Fin MacCool*, built especially for this work by the Osgood Dredge Co., of Albany, N. Y.

The hull proper is 120 ft. long, 40 ft. beam and 12 ft. 6 ins. deep. Secured at the forward end is a false bow or working platform, 10 ft. long, and aft there is a moulded false stern, 21 ft. long, added to make the dredge tow more easily and to furnish additional deck room, making the total length 151 ft. The dredge is always towed stern first. The A-frame and gallows-frame, forward, are 50 ft. high; the boom is 65 ft. long.

The usual manner of anchoring dredges in deep water is by anchor lines run out from the deck level; but as these lines would interfere seriously with the free handling of scows and tugs alongside, this dredge was supplied with two single spuds, each 3 ft. square, one at each end, which acted as attachments for the anchor cables by which the dredge was held in place. The bow spud extends 11 ft. below the bottom of the hull, the stern spud extends ordinarily 20 ft. below, but can be dropped to the bottom when the water is not more than 30 ft. deep, thus holding the dredge in place temporarily when running

cables. At the bottom of each spud there are three sheaves, two set across the spud and one fore and aft. On deck, close to the spud, are three other sheaves. The anchor cables pass from the drums over the deck sheaves, down the sides of the spuds, under the bottom sheaves, and out to the anchor buoys, stockless anchors being used.

This arrangement holds the dredge with anchors in five different directions, the lines being about 12 ft. under water, thus allowing scows and tugs to come alongside the dredge freely. See Fig. 23.



METHODS OF ANCHORING DREDGES.
FIG. 23.

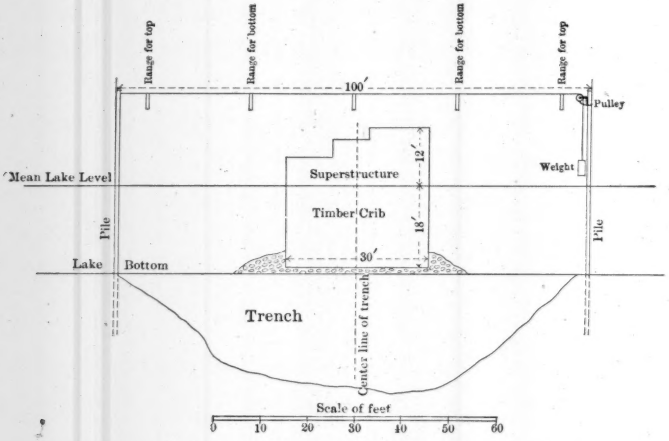
The dredge is moved by taking in or paying out the side and stern cables. The forward spud is raised by a cable attached to the hull, passing down through the longitudinal sheave and up over a sheave and thence to an auxiliary engine. The stern spud is raised by blocks and cables on the stern **A**-frame, the cables passing to the stern auxiliary engine.

The main engine, placed amidships, operates the dredge. It is an 18 x 24-in., double-cylinder engine with Stephenson reversing gear, and

is compound-geared to two 60-in. drums, with gear faces 12 ft. in diameter, and double frictions of the V-type, the male V's being of iron and the female V's lined with lignum-vitæ. The clam-shell bucket is of 10 cu. yds. capacity, and weighs 30 000 lbs.

There are also two secondary engines, one aft and the other forward, to handle the anchor and other lines. Steam is furnished by two Roberts water-tube boilers.

The trial performance of this dredge was made on March 23d, 1898, when starting work on the Stony Point section at Station 5. The hard bottom, mostly sand and gravel, which overlaid the soft clay, interfered somewhat with the progress at first. There was a little trouble,



SYSTEM OF RANGES.

FIG. 24.

also, with the hoisting cables, which had a tendency to run off the sheaves. The heating of the frictions also caused a little delay at first, especially when working in deep water, but this was soon remedied.

In the beginning, ranges to guide the work were placed on the shore and on the cribwork, which followed the dredging closely. As the trench advanced, a system of ranges, shown by Fig. 24, was used. This arrangement is the same as that used in the construction of the rubble mound. The location of the dredge, longitudinally, was determined by measuring distances from fixed piles, which were

usually driven at 400-ft. intervals, corresponding to Stations 0, 4, 8, 12, 16, etc.

This system of ranges was difficult to maintain, as the piles were not very stable in deep water, especially when subject to blows and shocks from passing tugs and scows.

Method of Work.—At first, the whole width of the trench was excavated by moving the dredge laterally, as required. In August, 1898, this method was changed. A cut, the width of the dredge, was made along one side, for 100 ft. or more, then another cut back through the center, and finally on the other side.

The dredge lay in working position all the time, except on Sundays and during violent storms, when it was towed to shelter. During ordinary storms it was allowed to tail off with the sea.

TABLE No. 15.—WORK OF THE DREDGE *Fin MacCool*, DURING THE SEASON OF 1898.

Began work March 23d, 1898; stopped work October 7th, 1898.

MONTHS.	Hours worked.	Hours delayed.	Working days.	Number of scows loaded.	Cubic yards of material dredged.	Cubic yards per hour worked.	Length of day's work, in hours.	Average load of scow, in cubic yards.	Average time to load one scow, in hours.	Average hours worked per day.	Average daily delay due to sea, in hours.
March.....	2¾	85¼	8	..	....	....	4 at 10 4 at 12	..	..	0.34	0.75
April.....	72	240	26	19	7 666	106.5	12	403	3.93	2.77[1]	1.81
May.....	107½	204½	26	45	14 665	136.4	12	326	2.39	4.13[2]	1.86
June.....	181½	162½	26	112	33 644	185.4	10 at 12 16 at 14	304	1.62	6.96	4.12
July.....	201	163	26	130	41 566	206.8	14	319	1.54	7.73	1.31
August.....	177½	200½	27	131	42 013	236.7	14	320	1.35	6.57	4.86
September.....	182½	181½	26	138	43 323	237.4	14	314	1.32	7.02	3.97
October.....	46¾	16¾	5¼	35	10 933	236.4	12	312	1.32	8.81[3]	0.57
Totals.	971	1 254	170¼	610	193 810	199.6	....	318	1.59	5.70	2.82
		1147.5[6]	161.4[5]							6.01[4]	

[1] 72 hours' delay in April, caused by wreck of dredge *Brian Boru*.

[2] 12 " " " May " " " " " " "
20 " " " " " " " " tug *Pete Gorman*.

[3] 2½ " " " October, foundations of timber crib.

106½ hours = 8.87 days.

[4] [5] This value, $6.01 = \frac{971}{161.4}$, 161.4 being the total time worked less time spent on the wreck of the tug *Brian Boru*, the tug *Pete Gorman*, and in leveling foundation timber cribs; and properly should not be charged against the work.

[6] This value, 1 254—106.5 hours (as above) is the net delay chargeable to the work. Hours worked plus hours delayed = 2 118.5.

For transporting the dredged material, dump scows, ranging in capacity from 250 to 500 cu. yds., were used. The distance to the dumping grounds varied from 8 000 to 11 000 ft. The round trip occupied generally about 1½ hours. The working day, during the summer months, was from 5 A. M. until 7 P. M., or 14 hours, with half an hour for breakfast and the same time for dinner. Late in the fall the working time was decreased to about 10 hours. The crew of the dredge consisted of one runner, eight to nine men and two cooks. The work of this dredge, for the season of 1898, is shown in Table No. 15.

The greatest quantity dredged in one day was 3 565 cu. yds. on August 30th, 1898, when twelve scows were loaded in twelve hours, an average rate of 297 cu. yds. per hour. The greatest rate for one scow on that date was 430 cu. yds. per hour.

During the season of 1898 the delays of the dredge *Fin Mac Cool*, on the Stony Point section, and their causes, were as follows:

General repairs	157½ hours.
Cables	88½ "
Heavy seas	480½ "
Anchors and placing dredge	124 "
Sheaves	37 "
Wreck of dredge *Brian Boru*	84 "
Frictions of engines	54½ "
Dump scows	55¼ "
Towing and coaling	25½ "
Wreck of tug *Pete Gorman*	20 "
Tug *Paddy Miles*	2 "
Clam	74½ "
Tugs	13½ "
Foundations of timber crib	2½ "
Holidays	14 "
Not known	20¾ "
Total	1 254 hours.

The short delay for breakfast and dinner each day is distributed among the other delays. The delay due to the wreck of the dredge *Brian Boru* and the tug *Pete Gorman* should be deducted, thus making the net delays 1 147½ hours.

The condition of the trench, while being excavated and after completion, was determined closely by soundings. The cross-sections were plotted as fast as the soundings were taken, and furnished a reliable record of the condition of the trench at all times.

When the plans were under consideration, a great deal of uncertainty existed as to the slopes at which the sides of the great trench excavated in the soft clay underlying the lake would stand. Opinions varied greatly. For purposes of estimate, the slopes were assumed at 1 on 1, and, for the standard crib, 36 ft. wide, it was intended that the trench should have a bottom width of 50 ft. It was found, however, much to the gratification of those in charge, that the slopes stood at much steeper angles than 1 on 1, and it was practicable, without increasing the quantity of excavation, to widen the bottom of the trench, this giving a greater stability to the structure. This trench excavation is an important and interesting feature of the work.

The dredging was completed on October 7th, 1898.

Cost of Dredging.— The total quantity of material excavated was as follows:

Season of 1897, by dipper dredges.....	8 117	cu. yds.
" 1898, " "	4 934	"
" 1898, by clam-shell dredge....	193 810	"
Total.....	206 861	cu. yds.

The contract price was 18 cents per cubic yard, scow measurement, making the cost $37 234.98. The place measurement of the trench is 192 258 cu. yds., showing an increase of 14 603 cu. yds. by scow measurement, or 7.6 per cent.

The following figures, showing the cost of the work to the contractor, are based upon data taken during the progress of the work, and may be assumed to approximate fairly the actual cost.

Assumed value of clam-shell dredge *Fin MacCool*.	$60 000
" " " dump scow *Protective Policy*....	12 000
" " " " " *Gold Standard*.....	10 000
" " " " " *Cuba Libre*.....	10 000
Total assumed value.....	$92 000

Annual depreciation, 10 per cent.
Annual interest, investment, 6 per cent.

Working days, Stony Point section	170¼
" " South Harbor "	41¾
Total	212

Proportion of depreciation and interest chargeable to:

Stony Point section	80.3 per cent.
South Harbor section	19.7 " "
Total	100.0 " "

Depreciation and interest for one year, $14 720, chargeable to:

Stony Point section	$11 820.16
South Harbor "	2 899.84

The following wages, as near as known, were paid during the season of 1898:

Runner	$75.00 per month.
Second runner	35.00 " "
Fireman, greaser and watchman	30.00 " "
Deck hands and cook	30.00 " "
Cook's help	12.00 " "
Extra men, without board	1.50 per day.

The cost of supplies per month for each man's board is assumed to be $12.00.

During the season of 1899, from careful records, the quantity of coal consumed per 100 cu. yds. of material excavated was found to be 0.31 ton. The cost was $1.50 per ton.

Assume 1.0 ton per 100 cu. yds. in April, 1898.
" 0.7 " " 100 " " May, "
" 0.35 " " 100 " during remainder of the season.

The expense of the siphoning-out scow *Faugh a Ballagh* was :

Engineman's wages	$40.00 per month.
Coal, 20 tons at $1.50	30.00 " "
Total	$70.00 " "

Superintendent's wages (half charged to dredge, and half to gravel filling)	$125.00 per month.
New wire cables.....	100.00 " "
Blacksmith shop.....	175.00 " "
Oil, waste, dope, etc.....	20.00 " "
Lines, hemp, cables, etc.....	40.00 " "
Miscellaneous expenses.....	50.00 " "
Yard expenses,.....	100.00 " "
Ranges, piles, etc., for the season.....	260.00 " "
Small tug, coaled, total cost.....	20.00 per day.
Large " " " "	25.00 " "
Large " " " "	30.00 " "

The operating expenses for the season, March to October, 1898, were then as follows:

Wages.....	$2 380.44	
Subsistence.....	907.38	
Fuel.....	1 212.00	
Siphoning.....	298.85	
Superintendence.....	384.62	
Wire cables.....	615.38	
Rope cables, lines, etc.....	246.15	
Blacksmith's shop.....	1 076.93	
Tugs, towing.....	6 505.00	
Yard expenses.....	615.38	
Ranges.....	260.00	
Miscellaneous.....	350.00	
Total operating expenses of dredge *Fin Mac Cool*.....		$14 852.13
Interest and depreciation.....		11 820.16
		$26 672.29

Quantity excavated by the dredge.....	193 810 cu. yds.	
Total cost per cubic yard.....	$\frac{\$26\,672.29}{193\,810}$	$13\frac{77}{100}$ cents.
Operating expenses, per cubic yard.....		$7\frac{67}{100}$ "
Interest and depreciation, per cubic yard....		$6\frac{10}{100}$ "

Trench Filling.—After the trench had reached the prescribed depth, it was filled with gravel. This material was dredged, by the *Brian Boru*, from the bed of the Niagara River, in the vicinity of the foot of Ferry Street, Buffalo, a short distance above the International Bridge. The dimensions of the *Brian Boru* are as follows: Steel hull, 95 ft. long, 28 ft. wide, 8-ft. sides; draft, 7 ft.; engine, 12 x 14-inch cylinders.

The first scow load of gravel was deposited in the trench on September 27th, 1897, between Stations 3 + 70 and 4 + 80. Five scow-loads were deposited during October, 1897, the last one on October 26th, 1897, a total of 2 478 cu. yds. during the season.

The two dump scows which were used for transporting this material are entirely of steel. Each is 175 ft. long, 25 ft. beam and has 8 pockets. The capacity of each pocket to deck level is 68.5 cu. yds. or a total of 548 cu. yds. They were not loaded to their full capacity, however, on account of the difficulty in towing up the rapid current of the river. The average scow load for the season of 1897 was 413 cu. yds.

The trench filling, which was suspended at the close of the season of 1897, was resumed on April 28th, 1898, when two small scowloads of gravel, containing, respectively, 60 and 64 cu. yds., were deposited. The regular depositing of gravel with the large scows was not resumed until May 18th, 1898, when one scow load was placed. During the season of 1898, two large wooden dump scows were also used for transporting gravel. Each is 175 ft. long, 28 ft. beam and contains eight pockets. The capacity of each pocket to deck level is 68 cu. yds., or a total of 544 cu. yds. During the months of July and October, 1898, some smaller scows were also used in transporting gravel. The total quantity of gravel deposited in the Stony Point trench is given in Table No. 16.

The back-filling with gravel followed the excavation as closely as practicable. This was done by dumping the material from the scows until the filling reached the level of the lake bottom or the prescribed grade. The gravel, in dropping through about 30 ft. of water, spread out over the trench without forming lumps or hollows. The position of each scow at the time of dumping was noted, and was usually determined by measurement from the piles set at regular intervals, as stated previously. The position of the scows was also determined

from small spar buoys, fixed at known distances and at right angles to the axial line.

TABLE No. 16.

Month.	Number of scows.	Cubic yards.	Average scow load, in cubic yards.
September, 1897	1	455.5	455.5
October, 1897	5	2 022.5	404.5
April, 1898	2	124	62.0
May, 1898	18	7 499	416.6
June, 1898	30	12 681	422.7
July, 1898	140	48 542	346.7
August, 1898	75	32 920	438.9
September, 1898	76	33 713	443.6
October, 1898	117	40 495	346.1
November, 1898	37	14 931	403.6
Totals and average	501	193 383	386.0

Soundings over the deposited material were also taken frequently, to determine its elevation and guard against putting in more than would reach above the original lake bottom.

The cost of 193 383 cu. yds. of gravel filling, at 13 cents per cubic yard, was $25 139.79.

Rubble Foundation.—The next step, after the completion of the gravel filling, was the building of the rubble foundation for the timber cribs.

In the beginning, the stone for this was brought mainly from the line of the Erie Canal, the contract not permitting the use of Canadian stone. It was brought to the work in canal-boats, and was deposited by being dropped over the sides, piece by piece.

As on the stone breakwater, the canal-boats were aligned carefully on the site of the work, and moved laterally across the same as the work progressed. The fixed grade below mean lake level being known, the elevation of the foundation was regulated by sounding continuously with a graduated pole, having an iron cross-piece at the bottom, to prevent it entering the interstices of the stone.

Later, in deeper water, the lower part of the foundation was built by dumping stone from dump scows, then topping out with stone deposited as mentioned above. In this later work, Canadian stone was used, a supplemental contract having been entered into, to allow its use,

The cost of the rubble foundation was as follows:

Foundation stone from United States	12 605.2 tons at 80 cents	$10 084.16
Foundation stone from Canada	6 363.1 tons at 75 cents	4 772.33
Rubble wall stone	202.5 tons at 80 cents	162.00
Totals	19 170.8	$15 018.49

Timber Cribwork.—As previously stated, the Stony Point breakwater is a timber-crib structure filled with stone, resting upon a rubble foundation, placed upon gravel filling.

The substructure consists of a number of cribs of different widths and depths, but of uniform length, except in one case. Where the depth of the water was 22 ft. or more, the so-called standard cribs, 60 ft. long, 36 ft. wide and 22 ft. deep, were used. In the shallow water, as the shore was approached, the width and depth of the cribs were reduced to correspond.

The superstructure varies in the different crib sections; the height of the banquette on the harbor side is uniformly 6 ft. above mean lake level; that of the parapet, for a distance of 420 ft. at the shore end, is 8 ft., the remainder being 12 ft. The dimensions of the different sections are shown in Table No. 17.

TABLE No. 17.

Cribs.				Superstructure.					
				Banquette.		Middle Deck.		Parapet.	
No. of section.	Width, in feet.	Depth, in feet.	Length, in feet.	Width, in feet.	Height, in feet.	Width, in feet.	Height, in feet.	Width, in feet.	Height, in feet.
1	16	4	300.0	8	6	None.		8	8
2	16	8	120.5	8	6	None.		8	8
3	24	12	360.8	12	6	None.		12	12
4	30	18	541.1	10	6	8	10	12	12
5	36	22	1504.5	12	6	12	10	12	12
Total..	.....	.....	2826.9	.....	.....	.....	.....	.....	.....

All heights are above water level.

Designs.—The cribwork is the same in general design as that heretofore used in the "old" breakwater, but several changes were made in the details, the most important being as follows:

Corner and Side Posts in Cribs.—Corner posts and side posts are screw-bolted to the crib walls and cross-ties. These posts do not extend up into the superstructure, but only to the top of the cribs, and are intended, principally, to strengthen the cribs below water level anticipatory to placing a concrete superstructure upon them in the future. The cribs are 60 ft. long, and posts are set in each corner, and two at equal spaces along the lake and harbor walls, making eight posts in each crib.

Grillage Planking in Cribs.—The grillage planking is confined to the middle third of the crib, leaving ample space for the stone filling to run out under the crib walls and support them in case of undermining.

Drift-Bolts in Cribs.—Button-head drift-bolts are used throughout, in place of headless bolts, for the timbers above the grillage bottoms.

Cross-ties in Cribs.—Longitudinal and alternate cross-ties are made up of three pieces of timber overlapping each other, and securely bolted, thus avoiding an unnecessary number of 36-ft. sticks and obtaining a diversified bill of timber.

Benches in Superstructure.—The superstructure in the former construction (after 1887) was 8 ft. high and 36 ft. wide, surmounted on the lake side by a parapet 4 ft. high and 12 ft. wide. In order to distribute the weight of stone filling more evenly, the superstructure is designed in three "benches," the first 6 ft. high and 12 ft. wide, the second 4 ft. high and 12 ft. wide, and the third, or parapet, 2 ft. high and 12 ft. wide. The banquette is thus lowered 2 ft., which reduces the likelihood of the guards of vessels moored to the breakwater "hanging" on the breakwater wall. The height of the structure above water on the lake side remains, as heretofore, 12 ft.

Deck Planking in Superstructure.—The deck plank is laid lengthwise with the breakwater, instead of crosswise, and there are walings along all edges, in order that mooring lines may not catch in the cracks and wrench off the planks.

Mooring Posts in Superstructure.—Iron mooring posts and cleats are used, instead of mooring rings, which are inconvenient for heavy lines. The posts and cleats are set alternately along the banquette, about

100 ft. apart. The posts are intended for use when the cleats may have been covered with ice and thus rendered unserviceable.

Timber.—All the timbers in the cribs are 12 ins. square, with the exception of the grillage plank and cross-tie supports, which are 6 x 12 ins. The length of the timbers varies from 7 to 36 ft. The cribs are of hemlock, except the three top courses and the upright posts, which are of white pine. Square butt joints are used in the side and end walls. At the corners the timbers are left with their full dimensions, the side and end timbers overlapping each other alternately.

The cribs consist of two essential parts: The grillage bottom, made up of three courses of timber firmly fastened by screw-bolts; and the remaining part, made up of nineteen courses, securely fastened by drift-bolts.

The side walls are connected by cross-ties, spaced at 10-ft. centers (except in the first bottom course, where they are at 5-ft. centers). Half the cross-ties are in one piece, the full width of the cribs, 36 ft., while those of the other half are in three pieces overlapping each other. The cross-ties are gained into the side walls. The gains are fan-shaped, 12 ins. wide at the outer, and 9½ ins. wide at the inner, end. The depth of the gains is 12 ins., 6 ins. being cut out of the so-called receiving, and 6 ins. out of the covering, course. The tie-heads correspond to these dimensions, and are the full depth of the timber.

There are two rows of longitudinal ties in each crib, spaced at 13-ft. centers. They are usually 22 ft. long, overlapping each other. The cross and longitudinal ties are spaced so as to permit the crossing of the timbers without cutting. At all intersections the two are fastened by drift-bolts 24 ins. long.

The grillage floor is made up of one row of 12 x 12-in. timbers extending the whole length (in the center) of the crib, and four rows of 6 x 12-in. plank. As first constructed, these planks were confined to the middle third of the cribs, that is, between the two rows of longitudinal ties. Although spaced with 1-ft. intervals, experience demonstrated that the stone filling did not pass through these narrow openings, but left large spaces entirely devoid of stone. Subsequently, two rows of planking, one on each side of the center longitudinal, were removed and placed one on each side of the outer, longitudinal. This gave two openings, the entire length of the crib each 3 ft. wide.

The bottom of the lower series of cross-ties is 6 ins. higher than that of the side and end walls, leaving a space of that depth under the same. This allowed more or less settlement, to avoid which, 6 x 12-in. pieces of timber were attached to the bottom of some of the ties, giving them equal bearing with the side walls.

The timbers of the side and end walls above the grillage are fastened with drift-bolts, 32 ins. long, driven in each course, passing through two whole courses, and entering 8 ins. into the third. The bolts are also arranged so that there are two in each course in each bay or panel formed by the ties. The bolts are driven at a slight angle, to prevent one striking another.

The posts in the corners and at the sides are 22 ft. long. The corner posts are screw-bolted to the crib walls, and the side posts to the crib walls and cross-ties.

All screw-bolts are 1¼ ins. in diameter and have two round steel washers, 5 ins. in diameter and ½ in. thick. The heads of the screw-bolts passing through the corner posts and end walls are countersunk, and, consequently, are 1½ ins. shorter than those passing through the corner and side posts and side walls. The countersinking allows the cribs to come into actual contact when sunk in position. In some few cases the upper screw-bolts passing through the side walls were also countersunk, to prevent being bent by straight-sided vessels moored alongside the breakwater. The heads of the screw-bolts fastening the grillage walls were also countersunk.

After each covering course was placed, a 24-in. drift-bolt was driven through it and through the tie-head into the receiving course.

As mentioned previously, a diversified bill of timber was secured by putting in the side walls timbers in even lengths of from 12 to 26 ft., the various lengths being placed in such sequence as to break joints well.

The arrangement of the cross- and longitudinal- ties was such as to form series of pockets or recesses, thus avoiding the continuity of straight walls, the projecting timbers forming rests for the interior stone filling.

Tifft Farm Framing Yard.—The cribs were built at a construction yard established by the contractors at the extreme south end of the Buffalo City Ship Canal, on property owned by the Lehigh Valley Railroad Company, and known as the Tifft Farm docks. Timber and supplies were delivered there by vessel and by rail.

The construction yard covered an area of about 600 x 200 ft., the length fronting on a slip of the City Ship Canal. Facing this slip was a dock, 300 ft. long and 30 ft. wide, supported by two lines of piles, one driven along the outer edge and the other down the center. These piles were capped with 12 x 12-in. timbers, with their inner ends resting upon the bank. The piles were spaced at about 9-ft. centers. Two timber fenders were bolted to the outer row. The floor of the dock was composed of 3-in. plank securely spiked. There were three derricks on this dock, two large ones for handling timber for crib construction, and a smaller one for loading scows with timber for the superstructure.

The launching ways were about 150 ft. south of the dock, and consisted of a skeleton timber structure, supported at the outer end by piles. Back of the launching ways were two timber platforms, one 16 x 56 ft., the other 12 x 34 ft., and between them there was a derrick for transferring timber from the platform to the work on the ways. Timber arriving by rail was unloaded by a derrick.

An office, a store-house, dwellings and other buildings completed the plant, to which was added, later, a building containing saw-mill, gaining and tie-head machinery.

The hemlock timber was cut in Elk and Jefferson Counties, Pa., sawed and shipped from Carrier Station, on the Buffalo, Rochester and Pittsburg Railroad, to Buffalo, and thence over the tracks of the Lehigh Valley Railroad to the framing yard. The pine timber came from Michigan, being shipped from Marquette to Buffalo by vessel.

The grillage bottom, composed of three courses of timbers and ballast floor, was constructed on the ways. The corner and side posts were also attached in place. The structure was then launched and the required courses of side and wall timbers added, as well as the interior ties.

The framing of crib No. 1, one of the smaller shore-end cribs, 4 ft. deep, 16 ft. wide and 60 ft. long, was begun on July 23d, 1897. It was built entirely of hemlock, and contained no posts.

At the end of the working season of 1897, seventeen cribs had been launched, of which five were 4 x 16 x 60 ft., one was 8 x 16 x 120 ft., six were 12 x 24 x 60 ft., and five were 18 x 30 x 60 ft. Twelve of these cribs were wholly completed, while five of the 18 x 30 x 60-ft. cribs were only partly completed, generally lacking the three top courses

of pine. During the working season about one crib per week was launched.

During the season of 1897, 640 120 ft., B. M., of timber were framed at the yard. The average working force was one foreman, two carpenters, eight laborers and one team with driver.

Work at the yard was resumed on May 16th, 1898. Crib No. 21, the last of the cribs of reduced section, was finished on June 24th, 1898. Crib No. 22, the first of the cribs of standard section, 22 x 36 x 60 ft., was launched on June 6th, 1898. It contains 82 584 ft., B. M., of hemlock, and 15 096 ft., B. M., of pine—a total of 97 680 ft., B. M. A summary of the bolts in this crib is given in Table No. 18.

TABLE No. 18.—Bolts in Crib of Standard Section.

Kind of bolts.	Number.	Length, in inches.	Diameter, in inches.	Weight of each, in pounds.	Total weight, in pounds.
Screw-bolts, with washers.	57	37	1½	18.70	1 065.90
" " " " .	154	26½	1½	15.12	2 323.48
" " " " .	44	25	1½	14.58	641.52
Drift-bolts.....	684	32	1⅛	8.96	6 128.64
" "	600	24	1⅛	6.70	4 020.00
Spikes.....	102	12	½	0.85	86.70
Total weight.....					14 271.24

The weight of each screw-bolt includes one nut and two washers. The washers were 5 ins. diameter and ½ in. thick.

The theoretical quantity of rubble required to fill one crib is 1 442.18 cu. yds.

Owing to insufficient depth of water in the Buffalo City Ship Canal to float the standard cribs, they were only framed nineteen courses high at the yard. The remaining three courses of pine were placed on the cribs after towing them to Stony Point.

Considerable work was done at the framing yard during the season of 1898. Thirty-three cribs were launched, of which four were of the reduced section, 18 x 30 x 60 ft., the others being of standard size. The quantity of timber framed and placed in the cribs at the yard during the season was 3 022 928 ft., B. M. Generally, two cribs at a time were under construction. The force was two foremen, thirteen to eighteen carpenters, from thirty to thirty-eight laborers, and three teams with drivers. During the busy part of the season two cribs per week were launched.

During 1898 two machines for cutting gains and tie-heads were installed, but it was late in September before they were in running order. Late in the fall the contractors also concluded to build the cribs in lengths of 180 ft., but with the usual crib bottoms. Three of these were bolted together, and the courses above them were made continuous. At the close of the season, one crib of this length was under construction, having been built nine courses high. These cribs were intended for the South Harbor section.

Work at the Breakwater Site.—The first crib was sunk on August 26th, 1897. Its dimensions were 4 x 16 x 60 ft., being one of five of that size. At the end of the season of 1897, six cribs had been placed, 180 ft. of superstructure had been completed and 240 ft. were partly finished.

A storm on November 6th, 1897, the wind blowing 50 miles an hour, moved the outer end of the unfinished work 1 ft. toward the harbor side. It is likely that the narrow width of the rubble foundation on that side aggravated this movement.

On April 11th, 1898, operations at the breakwater site were begun, and consisted in the rectification of the outer end of the work which had been displaced. After this had been rectified the superstructure was commenced, and on May 28th, 1898, 420.5 ft. of breakwater had been completed. The first crib of the season was sunk on May 25th, 1898, and others followed rapidly. The first standard crib, as well as the last crib of reduced section, the two being coupled together, were placed on July 26th, 1898. The last crib was sunk on December 3d, 1898, making a total of thirty-six cribs during the season.

All the cribs rest upon a foundation of rubble, which, for the standard cribs, is about 8 ft. deep. The lower part of this foundation was built by dumping stone from dump scows, after which it was brought up to the proper elevation by casting stone overboard from deck scows and canal-boats. The latter were usually moored along the inner or outer edge of the foundation, and then moved across gradually as the work progressed.

To attain the proper elevation of the stone, a workman with a sounding pole walked continually along the edge of the vessel, constantly testing the depth, due allowance being made for the variation from mean lake level, thus securing a comparatively level foundation.

In sinking a crib, the inner end was held in close contact with the

one previously sunk, the outer end being held in position by cables leading to large stone-anchor buoys and drawn taut by capstans placed temporarily on the cribs. The sinking was done by loading stone on a temporary deck on the crib, until it had sunk to 6 or 8 ins. above mean water level, as indicated by the water gauge. Stone was then thrown into the crib along the sides, in order to prevent further sinking.

It should be mentioned that the bottom of the crib is kept about 1 ft. above the top of the foundation, the latter being built to admit this. The tendency of the stones cast along the sides is to find their way under the side walls, forming a uniform support. Of course, this is mainly dependent on the size of the stone, small stone being preferable. The filling of the cribs proceeded simultaneously from both sides, vessels with stone being moored close along each side of the crib. The stones were cast in by hand. The alignment of the cribs was regulated by a transit on the completed work. In the case of the Stony Point breakwater, the lake face was used for the axis.

The construction of the superstructure was similar to that of the cribs, as far as sizes of timbers and arrangement of cross- and longitudinal- ties are concerned. The standard plan consists of three decks, designated, respectively, the banquette, middle and parapet decks. The first is 6 ft. high, the second 4 ft. high, and the third 2 ft. high, and each is 12 ft. wide, the total height above water on the lake side being 12 ft.

The cross-ties are spaced at 10-ft. centers, except those immediately supporting the different decks, where they are spaced at 5-ft. centers. The cross-ties of the lower part of the superstructure, or below the banquette level, were made up, alternately, of whole timbers and of three overlapping pieces of timber. The cross-ties of the middle deck were uniformly 24 ft., and those of the parapet deck 12 ft., long. The longitudinal ties were from 22 to 23 ft. long.

The deck planks were laid lengthwise with the structure, with 6 x 12-in. walings along all edges. The specifications required these planks to be 12 ins. wide, but, owing to the difficulty of getting such a large number of this width, the contractors were allowed to use plank 8 ins. wide on the parapet deck, 10 ins. wide on the middle deck, and 12 ins. wide on the banquette deck.

Iron mooring posts and cleats are set alternately along the banquette deck.

In December, 1898, the close of the season, there had been completed 2 345.4 ft. of breakwater on the Stony Point Section, and, in extension thereof, 240 ft. of cribs had been sunk in place on the rubble foundation, and were left exposed to the winter's storms with practically no protection.

The third season's operations were begun on May 8th, 1899. The finished portion of the section, exposed to the winter's storms, was found to be in perfect condition. The 240 ft. of cribs in extension thereof were also found in good condition, having been damaged only slightly at the outer lake corner, where the crib wall had been torn out, for 4 ft. in depth, and the stone filling washed out. This damage was thoroughly repaired and the cribs refilled with stone. On the completion of these repairs the work of setting the four cribs remaining to complete the required length, and the building of the superstructure, was commenced.

At the close of the fiscal year, June 30th, 1899, this superstructure was finished, completing the entire Stony Point Section. The length of the section is 2 802.9 ft., measured along the axis. An L at the outer end, as provided by the supplemental contract of September 26th, 1898, projects shoreward 24 ft. beyond the general harbor line, making the total length of the structure 2 826.9 ft.

TABLE No. 19.—Cost of Timber Cribwork.

Timber crib,		300.0	linear feet	at	$14.41		$4 323.00
"	"	120.5	"	"	19.65		2 367.83
"	"	360.8	"	"	41.27		14 890.22
"	"	541.1	"	"	67.48		36 513.43
"	"	1 504.5	"	"	92.37		138 970.67
		2 826.9					
Mooring cleats, 14 at $6.00							84.00
Mooring posts, 10 at $36.00							360.00
Extra work on L at north end							725.96
Repairs, contract of Charles A. Dennis							3 819.46
							$202 054.57
* Deduct 40 454.2 tons filling stone (Canada) at 5 cents							2 022.71
							$200 031.86

* This deduction was made on account of allowing the contractors the privilege of using Canada stone, in lieu of United States stone as per original contract.

TABLE No. 20.—Quantities of Material Used in the Stony Point Timber-Crib Breakwater.

No. of section.	Number of cribs.	Timber.		Screw-bolts, in pounds.	Drift-bolts, in pounds.	Spikes, in pounds.	Number of cleats.	Number of posts.	Foundation stone, in tons.	Filling stone, in cubic yards.
		Hemlock, in feet, B. M.	Pine, in feet, B. M.							
1	1–5	56 560	103 797	2 848	15 579	1 565	.....	.....	66	1 391
2	6	24 800	60 896	2 696	8 518	607	.....	.....	392	787
3	7–12	185 184	271 613	14 487	45 984	2 591	5	.....	3 633	5 162
4	13–21	559 548	514 626	35 197	117 116	5 099	3	4	5 027	13 000
5	22–46	2 064 600	1 583 149	101 620	380 446	16 761	6	6	9 850	50 911
Totals, 46		2 890 692	2 534 081	156 848	567 643	26 623	14	10	18 968	71 250

203 tons of rubble were also used for a stone wall at the shore end.

This breakwater withstood successfully the severe gales of December 5th and 12th, 1899, and also that of September 12th, 1900. The very severe gale of November 21st, 1900, however, caused some damage to the parapet deck, especially at the outer end. This damage consisted mainly of the breaking of the upper ties, which formed the direct support of the 3-in. longitudinal deck, and, consequently, the partial wrecking of this deck. The cause of the breaking of these ties was probably the excessive spans between the supports, namely, 10 ft.

In making the necessary repairs, some material changes were made in the parapet. These consisted in the removal of the 3-in. longitudinal deck and the substitution of a 6-in. deck laid transversely and supported by two rows of longitudinal stringers. Considerable blocking was also inserted between the ties, adding greatly to their bearing power.

South Harbor Section. Timber-Crib Breakwater.

Dredging.—The original plans for the extension to Stony Point provided for the construction of a 9 700-ft. section of breakwater, beginning at a point even with the south end of the old breakwater, but 250 ft. lakeward therefrom, and extending thence south in a straight line; the northern 5 000 ft. to be of the rubble-mound type and the remaining 4 700 ft. to be of the timber-crib type upon an artificial foundation extending down to solid rock.

The northern end of this main breakwater section is now known as the Stone Breakwater and the southern end as the South Harbor Sec-

tion. Before construction was commenced on the latter, a slight change was made in the South Harbor opening or entrance, and this, coupled with some minor changes, increased its length to 4 989 ft., the same as the stone breakwater, making the combined length 9 989 ft.

While work was in progress on the South Harbor section, the Chief of Engineers, on August 21st, 1899, approved the recommendation of Major Symons to reduce its length, and to increase the length of the stone breakwater accordingly. This added 2 250 ft. to the rubble-mound section, and reduced the length of the South Harbor section correspondingly, making it 2 739 ft. long.

As stated elsewhere, the excavation of the Stony Point trench was completed on October 7th, 1898, on which date the clam-shell dredge *Fin MacCool* commenced work on the South Harbor section.

The ranges for keeping the dredge on the alignment had been erected a short time before, and consisted of pile clusters, with cables stretched between and range boards hung from them. The cables were placed only 7 ft. above the water surface, as it was not necessary to run tugs between them. The ranges are shown by Fig. 24. There was more or less trouble in the maintenance of these ranges, caused by storms and heavy seas washing out the pile clusters, and they were replaced by single piles, and these also failed. After they became so completely broken down that they could not be maintained, only a single cut, 40 ft. wide, was made along the center line.

A great deal of time was lost, owing to the heavy seas, and work was finally abandoned for the season on November 24th, 1898, the weather becoming very cold and stormy.

Up to this time, 41 118 cu. yds. of material had been dredged, the depth of the excavation averaging 50 to 55 ft. below mean lake level. No part of the trench, however, was completed at this time.

The principal delay was caused by the frequent storms. On October 22d, 1898, a gale arose so suddenly that in twenty minutes there was a raging sea, the waves running even with the deck. The work of letting go the anchors was commenced immediately, but took much time, and the stern cable had to be cut. The attending tug not being able to handle the dredge, it drifted rapidly toward shore, and its beaching was only prevented by dropping the bow spud to the bottom when in the breakers about 1 000 ft. from shore. At this moment

assistance came from the largest harbor tug in Buffalo, and the dredge was towed to shelter. Fortunately, little damage had been done.

Work for the season of 1899 was resumed on the trench on May 5th. The ranges suspended from wire cables were abandoned, and piles were driven instead, back ranges being erected on the Stony Point breakwater.

The method of dredging was as follows: A lake-side cut, about 100 ft. long and 35 to 40 ft. wide, was first made, then the dredge was shifted to the middle and worked forward the same distance, then shifted to the harbor side and worked backward, as on the lake side, thus making three parallel cuts. Several hundred feet of trench having thus been excavated, the dredge was moved to the point of beginning and the bottom of the cut thoroughly cleaned up.

After the first crib on the South Harbor section had been placed in position it was used for the emplacement of the back ranges. The longitudinal location of the dredge was determined by measurement from fixed piles, as on the Stony Point section, and also from fixed spar buoys, set to one side at right angles.

Three steel scows were used to transport the excavated material, and one wooden scow was used for a few days.

The general dimensions of the tug *Shaughraun*, used in towing the dredged material to the dumping grounds, off the mouth of Smoke's Creek, are: Length, 54 ft.; beam, 14 ft.; draft, 6 ft.; gross tons, 29; net tons, 14; iron hull; wheel, 54 ins.; engines, 12 x 15-in. cylinders; boiler, 8 ft. long and 51 ins. diameter; speed, 9 miles per hour, light. The tug *Spalpeen* was a general tender for the dredge, and also placed the gravel scows ready for dumping.

As at Stony Point, the dredge lay at anchor, in working position, throughout the week, and on Sundays was towed to shelter, behind the completed Stony Point breakwater. After September 18th, 1899, the dredge was towed to shelter behind the partially completed South Harbor breakwater every night.

Coal was brought from Buffalo on a small deck scow, provided with derrick and hoisting engine, and was transferred to the dredge in the evening by men who had been dumping gravel scows during the day.

The daily routine on the dredge was as follows: Men rise at 4.30 A. M.; dredge begins work at 5.00; breakfast, 6.00 to 6.30; dinner 12.00 to 12.30; supper, after finishing work, at 7.00 P.M.

Delays and Accidents.—The principal cause of delay was the sea and wind (about three-eighths of the total delays). Considerable delay was due to the clam. The sprocket chain connecting the traveling shaft and main shaft of the clam was shortened too much during the winter of 1898–99, in repairing the clam, so that, when it was dropped on the rock bottom, the shock was taken up by the chain and connections, instead of being transmitted to the top of the frame by the corner rods. This resulted in breaking the sprocket chain, and also the connection on the traveling shaft, several times. This was repaired temporarily by wrapping cables around the shaft, until new castings could be obtained. The traveling shaft broke several times and the main shaft once, but the latter was due to the bearings having become worn. The port, outer corner rod was badly bent several times and broken once or twice, due to striking a scow. Toward the end of the season the clam had become so badly worn, and was in such poor condition, that several days were spent in repairing it. At the end of the work it was almost a wreck, and one or more days' work, cleaning off the rock bottom, would have made necessary its thorough repair.

One of the main cables, on the starboard side, was in use for nearly a year, and was finally taken off because of general breaking and pulling apart. The work during 1899 was harder on the clam, apparently because of getting closer to and striking the smooth rock bottom, which made it necessary to repair continually. The dump scows were kept in much better condition than during previous seasons.

Material.—Most of the material dredged was a moderately stiff red clay, mixed with some blue clay, which, in some places, seemed somewhat harder to dig than in others. Overlying this clay was a layer of sand, perhaps 1 or 2 ft. thick. Underlying the clay, next to the rock, was a layer of hard blue clay, mixed with broken stone, or gravel, and, in places, there were a good many large boulders. The thickness of the layer next to the rock varied greatly. In some places it was so great that nearly a scowload was dredged from a small area, but for a good portion of the trench it was very thin, so that the rock was dredged off quite clean.

Dumping Grounds.—The dredged material was dumped in front of the lake face of the Stony Point breakwater, up to May 10th, 1899, after which only the last scow each day was dumped there, and an occasional scow during the day, in case of necessity. It was dumped

about a scow's width from the wall and was distributed along the whole length, with the greater portion as near the shore as possible. From May 11th to 18th the material was dumped opposite Smoke's Creek, and from May 19th to 26th opposite Rush Creek, 1 550 ft. from shore. This locality was hard to reach, and on May 26th the buoy was moved to a place opposite a point on shore 1 500 ft. south of Smoke's Creek and 1 200 ft. from shore, about 6 500 ft. from the Stony Point triangulation station, the material being dumped on the shore side of the buoy. The distance of this buoy from the dredge on May 26th was 10 000 ft., and the time for the round trip was 1 hour and 6 minutes. On October 16th the distance was 12 300 ft., and the average time for the round trip 1 hour and 10½ minutes.

The trench was completed on October 16th, 1899.

Cost of Dredging.—The total quantity dredged in 1898 and 1899 was 357 461 cu. yds., and the cost, at 18 cents per cubic yard, was $64 342.98.

During October and November of 1898 and the season of 1899 the delays of the dredge *Fin MacCool*, on the South Harbor section, and their causes were as follows:

Clam	103	hours	30	minutes.
Main engines	5	"	55	"
Anchors and attachments	42	"	35	"
A-frame and boom	15	"	15	"
Miscellaneous breakdowns	2	"	10	"
Waiting for scows	30	"	54	"
Meals	147	"	10	"
Fog and rain	6	"	15	"
Sea	524	"	15	"
Mooring and placing dredge	114	"	19	"
Miscellaneous	65	"	21	"
Hoist cables	24	"	53	"
Boilers	1	"	05	"
Leaking scows	1	"	05	"
Repairs to scows	14	"	51	"
Siphoning scows	3	"	47	"
Waiting for tugs	9	"	37	"
Holidays, etc	42	"	53	"
Total	1 155	"	50	"

TABLE No. 21.—Work Done by the Dredge *Fin MacCool*, on the South Harbor Section, During 1898 and 1899.

Year and month.	Time worked.	Time delayed.	Number of days.	Length of day, in hours.	Number of scows loaded.	Average load of scows, in cubic yards.	Average number of hours to load scow.	Material dredged, in cubic yards.	Cubic yards per hour worked.	Average hours worked, per day.	Average daily delay, in hours, due to sea.	Average number of cubic yards per day.	Maximum day's work, in cubic yards.	Maximum hour's work, in cubic yards.
1898.	h. m.	h. m.												
October.....	91 00	158 00	20¾	12	82	316.4	1.11	25 945	285	4.38	3.81	1 250	3 062	354
November.....	54 45	197 15	21	12	53	286	1.03	15 173	277	2.61	6.07	722	2 834	362
Season of 1898..	145 45	355 15	41¾	12	135	304.6	1.08	41 118	282	3.49	4.95	985	3 062	362
1899.														
May.....	190 37	131 23	23	14	166	322.2	1.1	53 494	280.6	8.3	2.7	2 325.8	4 621	369
June.....	232 11	131 49	26	14	191	335.2	1.2	64 022	275.7	8.9	2.8	2 462.4	4 318	346
July.....	211 40	152 20	26	14	180	306.3	1.2	55 138	260.4	8.1	1.6	2 120.7	4 001	364
August.....	255 31	122 29	27	14	217	358.7	1.2	77 848	304.7	9.5	1.6	2 883.2	5 037	438
September.....	133 12	206 48	24	14 / 12	109	335.7	1.2	26 597	274.7	5.6	3.8	1 524.9	3 804	344
October.....	98 02	55 46	13	12	89	328.5	1.1	29 244	298.3	7.5	0.5	2 249.6	3 510	350
Season of 1899..	1 121 13	800 35	.....	.....	952	332.3	1.18	316 343	282.1	8.0	2.3	2 262.5	5 037	438

The quantity of material excavated during the season of 1899 was 316 343 cu. yds., which, at 18 cents per cubic yard, cost $56 941.74.

Some figures, as to the cost of this work to the contractor, may prove of interest. As far as known, the wages paid by the contractor during the season of 1899 were as follows:

Dredge runner	one	at	$90.00	per month.
Second dredge runner	"	"	35.00	" "
Fireman	"	"	35.00	" "
Deckhand	"	"	35.00	" "
Greaser	"	"	30.00	" "
Watchman	"	"	30.00	" "
Deckhands	seven	"	30.00	" "
Cook	one	"	30.00	" "
Cook's helper	"	"	15.00	" "

For working overtime the men received 15 cents per hour, the runner 30 cents per hour. The supplies for board cost $12 per month, per man.

From a careful record of the coal consumed, the quantity per 100 cu. yds. of material excavated was found to be 0.31 ton, but 0.35 ton is assumed for computations. Bituminous coal was used, and cost $1.50 per ton. The cost of the siphoning-out scow *Faugh-a-Ballagh* was:

One man, per month	$40.00
Coal, 20 tons at $1.50	30.00
Total	$70.00

Half the superintendent's wages ($125.00 per month) was charged to the dredge, the other half to gravel filling	$62.50	per month.	
New cables	100.00	"	"
Oil, dope, waste, etc.	20.00	"	"
Blacksmith's shop	175.00	"	"
Lines, cables, etc.	40.00	"	"
Miscellaneous expenses	50.00	"	"
Yard expenses	100.00	"	"

Ranges and piles for the season.	12 piles	at	$6.00	$72.00
Driving	12 "	"	12.00	144.00
Cross-ranges,	10 "	"	2.00	20.00
Back-ranges, piles, etc.				20.00
				$256.00

Small tugs, coaled	$20.00 per day.
Larger " "	25.00 " "
Large " "	30.00 " "

The assumed value of the plant is as follows:

Clam-shell dredge, *Fin MacCool*	$60 000.00
Steel dump scow, *Protective Policy*	12 000.00
" " " *Gold Standard*	10 000.00
" " " *Cuba Libre*	10 000.00
Total	$92 000.00

Annual depreciation, 10%; and interest, 6%; total, 16%, and, for one year, $14 720.

ESTIMATED COST.

Board	$980.34
Coal	1 660.80
Siphoning scow	369.24
Superintendence	329.67
Cables, main	527.49
Lines, ropes, etc	210.99
Blacksmith's shop	923.11
Yard expenses	527.49
Tugs	5 830.00
Range piles	256.00
Miscellaneous	263.74
Wages	2 823.14
Total	$14 702.01
Depreciation and interest	14 720.00
Total	$29 422.01

Cubic yards excavated ..... = 316 343

Cost per cubic yard ..... $= \frac{\$29\,422.01}{316\,343} = \$0.09\frac{30}{100}$.

Operating expenses ..... $= \frac{\$14\,702.01}{316\,343} = 0.04\frac{64}{100}$.

Interest and depreciation .... $= \frac{\$14\,720.00}{316\,343} = 0.04\frac{65}{100}$.

Trench Filling.—The filling of the trench with gravel was begun on May 18th, 1899, on which date five scowloads, containing 2 072 cu. yds. of gravel, were deposited. Most of the gravel was dredged

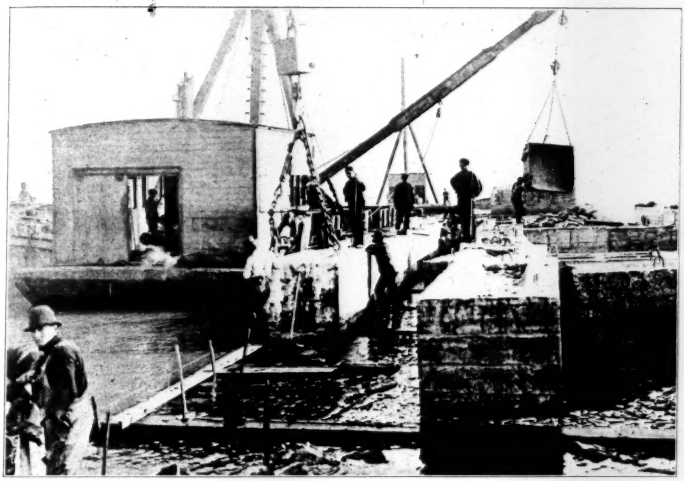


FIG. 1.—SOUTH HARBOR SECTION. SETTING CONCRETE BLOCKS ON HARBOR SIDE. 1901.

FIG. 2.—SOUTH HARBOR SECTION. CONCRETE SUPERSTRUCTURE. PARAPET WALLS IN PLACE. 1901.

from the bed of the Niagara River, loaded in dump scows and then towed up the rapid current of the river, by the tugboat *Charles F. Dunbar*, built especially for this work. This tug towed the loaded scows to the head of the river, from which point they were towed to the work by a smaller tug, dumped and taken back, the larger tug towing the empty scows down the river. The average day's work was five scows, each with an average load of 465 cu. yds.

The supply of gravel from this source being insufficient for the proper progress of the work, the contractors were allowed to obtain gravel from the bed of Lake Erie, just off the shore in the vicinity of the sand-catch pier, and began digging there on July 3d, 1899. Work was suspended on July 10th, 1899, after taking out fifteen scowloads, or 3 858 cu. yds. The gravel obtained here was rather fine, occasionally with some silt, and was not as good as that from the Niagara River. For this reason, it was deposited in the deepest part of the trench.

The length of the South Harbor section having been curtailed, as before stated, the contractors were anxious to complete it before the close of the season, and they were again allowed to dig sand near the sand-catch pier. Accordingly, a ladder (or elevator) dredge began work on September 1st, 1899, and continued through September and October, 1899. Owing to the accumulation of silt at places, there was much trouble in obtaining proper material. As before, it was placed as low down in the trench as possible.

The cost of 321 695 cu. yds. of filling, at 13 cents, was $41 820.35.

Rubble Foundation.—The foundation for the timber cribs was built in the same manner as that for the Stony Point section, except that more stone was deposited by dump scows, a difference in the conditions allowing this to be done. The lake face of this breakwater was also rip-rapped to a greater height above the lake bottom, which accounts for the larger quantity of stone, as compared with the Stony Point section.

The cost, up to December 15th, 1902, was as follows:

Rubble from United States....	25 678.5 tons,	at $0.80	=	$20 542.80
" " Canada.....	87 907.8 "	" 0.75	=	65 930.85
" " Canada.....	12 579 8 "	" 0.72½	=	9 120.36
Total.....	126 166.1			$95 594.01

Timber Cribwork.—The timber-crib breakwater, which is part of the project of the extension to Stony Point, adjoins the stone breakwater on the south, being a prolongation of the latter, with the exception that, owing to the different cross-sections of the two structures, the axes are not coincident, although parallel.

The breakwater is 2 739 ft. long, measured along the lake face. It consists of sixteen cribs, one of which is 60 ft. long, 36 ft. wide and 22 ft. deep, and is placed at right angles to the axis of the breakwater, projecting 24 ft. beyond the general harbor face of the work, thus forming an L, to be used as an emplacement for a lighthouse, protecting the South Harbor entrance.

The remaining fifteen cribs are each 180 ft. long, 36 ft. wide and 22 ft. deep, differing in this respect from the Stony Point breakwater, where the cribs are each 60 ft. long, with the exception of one shallow crib, 120 ft. long.

The 180-ft. cribs are composed of three grillage bottoms, each 60 ft. long, 36 ft. wide and 4 ft. high. These bottoms are bolted together firmly at each of the two joints, by seven 1¼-in. screw-bolts, 26½ ins. long, making a composite grillage bottom 180 ft. in length. The remaining eighteen courses are then laid up in the usual manner, great care being taken to break joints properly. The arrangement of the cross- and longitudinal- ties, and vertical posts, was identical with that of the 60-ft. cribs. There was no difficulty in sinking these long cribs; in fact, they were placed and sunk more readily than the shorter ones, and an almost absolute alignment was obtained. A decided advantage over the shorter cribs is the obliteration of two joints, which is quite a consideration.

Subsequent experience, however, demonstrated that these cribs should have been built with at least two solid transverse bulkheads, or, better still, with five, one for every 30 ft. Such bulkheads would have resisted the tendency of the long cribs to "warp" at the top, due to the buffeting of heavy storms.

The first of these 180-ft. cribs, designated as Nos. 48–49–50, was begun early in November, 1898. At the close of the season, December 9th, 1898, this crib had been built nine courses high.

Work was resumed on May 4th, 1899, and the crib was sunk in position on August 14th, 1899, forming the first 180-ft. crib of the substructure of the South Harbor section. Crib No. 47, known as the L-crib, was sunk on July 20th, 1899.

The 180-ft. cribs contain the following quantities:

Hemlock						233 328	ft., B. M.
Pine						41 304	" "
Total						274 632	" "
Screw-bolts,	171	1¼ x 37	ins.	at	19.51 lbs.	3 336.21	lbs.
"	472	1¼ x 26½	"	"	15.85 "	7 481.20	"
"	44	1¼ x 25	"	"	15.33 "	674.52	"
"	8	1¼ x 19	"	"	13.25 "	106.00	"
Drift-bolts,	1 620	1⅛ x 32	"	"	9.01 "	14 596.20	"
"	1 856	1⅛ x 24	"	"	6.76 "	12 546.56	"
"	462	½ x 12	"	"	0.85 "	392.70	"
Total						39 133.39	"

The weight of the screw-bolts includes one nut and two washers with each bolt. The washers are 5 ins. in diameter and ½ in. thick.

The season of 1899 was one of great activity. Nine 180-ft. cribs were built and sunk and the superstructure completed over them. In addition, five 180-ft. cribs were built nineteen courses high, which included all the hemlock timber; another was built six courses high. At the close of the season 1 682.2 ft. of timber-crib breakwater had been completed.

Tifft Farm Framing Yard.—During the summer of 1898 the contractors installed two machines to lessen the labor of framing, a gaining machine and a tie-head machine, both manufactured by the Lake Shore Iron Works, of Marquette, Mich., for the Holley Manufacturing Company. They were housed in a building called the mill.

The mill was a wooden building, near the middle of the framing yard, just off the construction dock, 36 ft. wide and 60 ft. long. The boiler-house, 14 x 28½ ft., was at one end of the mill. The building also contained a butt-off machine, operating a 30-in. circular saw. The engine-room, 13½ x 15½ ft., was in one corner of the mill, and contained a Westinghouse engine. Adjoining the engine-room was a room, 12 ft. long, containing a Westinghouse air compressor. The air tank was outside of the mill.

Adjoining the air-compressor room there was an 8 x 12-ft. tool-room. In the central part of the mill were rollways on which the

timber was carried from the circular saw to the gaining and tie-head machines. A derrick, near the boiler-house, handled the timber on a runway, carrying it to the circular saw for squaring up. Outside the mill and just off the framing machines were two other rollways on which the framed timber was handled.

The manipulation of the timber at the yard and mill was as follows: The timber was dragged to the mill by horses. As a rule, there were two teams most of the time. It was delivered at the west end of the mill, squared off by a carpenter, and then hoisted by a small hand-derrick and placed on a runway on which it was propelled to the butt-off machine, where a circular saw squared the ends. The number of the stick, indicating its position in the crib, was marked on its end, and it was then passed over the rollway to the gaining machine, which cut the gains. The timber was then passed out of the mill on the adjoining runway, where a carpenter removed any remaining imperfections, caused by the varying thickness of the timber. It would have been a great advantage had the timber first been run through a planer or sizing machine, as is the practice on some other works.

The cross- and longitudinal- ties were manipulated in the same manner as the wall timbers, except that, instead of going to the gaining machine, they were run to the tie-head machine. After passing through the latter the stick was ready to put in the crib. The tie-head machine cuts two tie-heads at one time, of course on two different sticks. After the timber was framed it was loaded on small cars, running on a 3-ft. track, and taken to the dock, sorted and placed in the work. On the dock it was handled by derricks to the crib being built.

Obviously, the crib bottoms were constructed first. They were built on launching ways at the water's edge just east of the construction dock. The timber was handled by a derrick located between two platforms on which it was stored.

In building the grillage bottom great care was taken to make it perfectly square. This was usually done by measuring the calculated diagonal with a steel tape. After three courses had been placed and fastened together by screw-bolts, the vertical posts were put in and the bottom was ready for launching.

After three cribs had been launched they were bolted together and

the composite structure towed in front of the construction dock, where the remaining courses were fastened in place.

For boring the bolt holes the contractors used a "Little Giant" pneumatic boring machine, made by the Standard Pneumatic Tool Company, of Chicago, Ill.

A 60-ft. crib bottom was launched about every four days; and a 180-ft. crib was built nineteen courses high about every two weeks.

Superstructure.—After the cribs had been sunk and filled with stone, at least fourteen days were allowed for them to settle fully upon their foundation before the building of their superstructure was commenced.

This superstructure is continuous over the cribs. On this section it covered three 180-ft. cribs, making sections each 540 ft. in length, except the first section, which also covered the L-crib, making it 576 ft. long. These are the theoretical lengths, the actual length of a section being somewhat more, owing to the impossibility of making actual contact of the cribs. Consequently, there are five separate sections, one of ± 576 ft. and four of ± 540 ft. each, making a total length of 2 739 ft.

The superstructure was built as follows: The cribs were first leveled to the required grade above mean lake level, either by building up with timber when too low, or by adzing down when too high. The lake side was always kept about 4 ins. higher than the harbor side, to allow for settlement, the greater weight being on that side.

The side-wall timbers were then placed and drift-bolted. The first course in each set, known as the receiving course, had all been squared and the gains cut at the framing yard. The ties were then placed in position and the so-called covering course put on, this course being framed on the breakwater site. The other courses followed in regular sequence until the top was reached, the proper bolts being driven as the work progressed. The stone filling was generally put in as the timber work advanced, although there were exceptions, as the rubble was sometimes slow in arriving. The decking was the last work to be done.

A small deck scow, rigged with derrick and hoisting engine, handled the timber on the work.

The rubble was transferred from deck scows and canal-boats by one of the floating derricks used in the adjacent rubble-mound section.

The building of the superstructure was very expeditious. Section No. 1, 577.1 ft. long, was completed on October 31st; Section No. 2, 540.4 ft. long, on November 30th; and Section No. 3, 540.7 ft. long, on December 19th, 1899.

Several severe storms occurred during the early part of December, 1899. The gale of December 12th, was especially severe, and caused considerable damage to Sections Nos. 1 and 2. The damage was confined to the parapet or upper deck, and seventy ties were broken. These were 12 x 12-in. ties, 12 ft. long, over all, and 10 ft. between supports, spaced at 5 ft. from center to center. They failed by breaking in the middle. The breaking of these ties also caused the failure of some of the 3-in. decking, which was placed at right angles to the ties and was supported by them. Undoubtedly, the damage was caused by the impact of the falling water of the high seas, which rose to a great height upon striking the vertical face of the lake side of the breakwater. Section No. 3 was in course of construction at the time of the gale, and, in order to strengthen the unfinished portion of the parapet, a 12 x 12-in. timber was placed longitudinally with the axis in the center of the parapet and resting upon the last but one series of ties, thus affording an additional support for the upper ties and deck.

The completed timber-crib breakwater withstood the winter of 1899–1900 admirably, except that there was some little movement of Crib Nos. 57–58–59, due to the enormous ice pressure.

Work for the season of 1900 was commenced on May 17th. A slight change was made in the parapet of the remaining part of the timber work, which consisted in spacing the top row of ties at 3 ft. 4 ins. from center to center, instead of at 5 ft., as called for by the original plans. A 6 x 12-in. longitudinal timber was also placed under these ties as an additional support. These changes were made on account of the weakness developed in the parapet laid during the season of 1899.

A very severe gale occurred on September 11th and 12th, 1900, following closely upon the hurricane which wrecked the City of Galveston, Texas, and did considerable damage to the completed work. The parapet suffered most, although more or less damage was also done to the middle deck. The damage consisted in the breaking of the ties, the resulting deformation of the decking, and the breaching of the upper timber walls.

Some damage was also done to the work under construction on Section No. 5, which was repaired at once by the contractors. The

remaining part of the unfinished work was strengthened by the addition of longitudinal timbers, blocks and struts. Steps were taken at once to repair the damaged work, and a large quantity of timber was accumulated for this purpose. A working plant and force was organized, when, on November 21st, 1900, while the repair work was being prosecuted, another gale occurred. This gale was of unusual violence, the wind blowing from the west at the rate of 80 miles an hour.

Considerable damage was done to the superstructure of the completed work, and more especially on Sections Nos. 1 and 2; on the former, 360 ft., and on the latter, 540 ft., were razed almost to the water's edge. Sections Nos. 3 and 4 were also badly damaged, the entire middle deck on the latter being razed to the banquette level. Where the ties had been spaced at 3 ft. 4 ins. from center to center, and extra longitudinal timbers inserted, the parapet withstood to some extent the effect of the gale. On Section No. 5, about 150 ft. of the work was badly damaged, the remainder standing very well. The substructure cribs suffered no damage.

The cost of the cribwork was as follows:

Cribwork, 2 763 lin. ft. at $92.37	$255 218.31
Mooring cleats, 15, " 6.00	90.00
Mooring posts, 15, " 36.00	540.00
Extra work on L at south end	725.96
Additional timber and bolts, strengthening parapet	1 367.66
	$257 941.93
Deduct for 52 636.2 tons Canadian filling stone, at 5 cents	2 631.81
Total cost	$255 310.12

Table No. 22 shows in detail the various quantities of material used in the construction of the South Harbor timber-crib breakwater up to December 15th, 1902.

The following statement gives the cost (to the contractor) of labor employed in building the timber cribs and superstructure of parts of the Stony Point and South Harbor breakwaters for the working season of 1899, being from May 1st to December 19th, inclusive.

The quantity of hemlock placed in cribs during the season was 3 207 144 ft., B. M., being wholly used in the South Harbor section. All the hemlock used in the cribs, and also the pine posts and an occasional course of pine, were framed and placed at Tifft Farm.

TABLE No. 22.

Number of section.	Number of cribs.*	Timber.		Screw-bolts, in pounds.	Drift-bolts, in pounds.	Spikes, in pounds.	Number of cleats.	Number of posts.	Foundation stone, in tons.	Filling stone, in cubic yards.
		Hemlock, in feet, B. M.	Pine, in feet, B. M.							
1–5	16	3 582 504	2 846 085	178 956	653 479	33 990	15	15	126 166	94 051

* Fifteen cribs, each 180 ft. long, one crib, 60 ft. long, set crosswise.

The quantity of pine used in the cribs and worked at Tifft Farm was 97 824 ft., B. M., making a total, of hemlock and pine, of 3 304 968 ft., B. M.

The cost of labor on this work for the season averaged $1.90½ per 1 000 ft., B. M.

All the hemlock, and the major part of the pine, was framed by machinery, one machine cutting the gains of the side- and end-wall timbers and another cutting the tie-heads fitting into these gains.

The quantity of timber worked by the mill, and handled in the yard, for the season was:

Hemlock	3 207 144 ft., B. M.
Pine	2 163 556 " "
Total	5 370 700 " "

The figures include some of the longitudinals and short ties which required no framing, also the covering courses of the superstructure, which were squared by the mill, but the gains were cut by hand on the work. On a basis of all the timber handled during the season, *viz.*, 5 370 700 ft., B. M., the cost per 1 000 ft., B. M., was 92 cents. The handling included unloading the timber from cars, piling it, transporting it to the mill and from the mill to the dock and to scows.

The cost per 1 000 ft., B. M., for this handling was 48½ cents. The quantity of timber framed and placed in the work at both breakwaters during the season was as follows:

Stony Point: Cribs	31 968		
Superstructure,	397 831	429 799 ft., B. M.	
South Harbor: Cribs	308 856		
Superstructure,	1 327 077	1 635 933 "	"
Total		2 065 732 "	"

FIG. 1.—SOUTH HARBOR SECTION. VARIOUS STAGES OF CONCRETE SUPERSTRUCTURE. 1901.

FIG. 2.—SOUTH HARBOR SECTION. COMPLETED CONCRETE SUPERSTRUCTURE. 1901.

The average cost of doing this work was $3.23 per 1 000 ft., B. M.

The total quantity of timber handled, framed and placed during the season was 5 370 700 ft., B. M., at an average cost of $3.82 per 1 000 ft., B. M., the handling and driving of bolts being included.

The average run of timber per linear foot of standard section is 1 526 ft., B. M., for the cribs and 772 ft., B. M., for the superstructure, or a total of 2 298 ft., B. M.

At the rate stated, $3.82, the cost of labor per linear foot of completed timber-crib breakwater was $8.78. This, of course, does not include the stone filling.

The weight of bolts and spikes of all kinds used per linear foot of breakwater was 217.4 lbs. for the cribs and 91.2 lbs. for the superstructure, a total of 308.6 lbs.

The rubble filling per linear foot required for the cribs was 27.2 tons and for the superstructure 11.0 tons, a total of 38.2 tons.

The wages paid at the Tifft Farm framing yard were as follows:

Superintendent	$100.00	per month.
Time keeper	65.00	" "
Watchman	50.00	" "
On cribs:		
Foremen	0.22½	per hour.
Carpenters	0.20	" "
Laborers	0.15	" "
In yard:		
Foreman	60.00	per month.
Laborers	0.15	per hour.
Teams and drivers	0.35	" "
Mill:		
Foreman	75.00	per month.
Engineman	65.00	" "
Machine men—framing	0.25	per hour.
Carpenters	0.20	" "
Laborers	0.15	" "
Team and driver	0.35	" "

Concrete Superstructure.—Soon after the demolition of the timber superstructure of the South Harbor section, several plans for a new

superstructure were considered, and it was finally decided to rebuild it with a concrete shell and fill the interior with rubble.

The necessary plans and specifications were at once prepared, and on January 25th, 1901, proposals for building 1 800 ft. of concrete superstructure were invited, to be opened on February 25th, 1901. In response, seven bids were received, all of which were rejected.

New bids for the work were asked on March 21st, 1901, to be opened April 5th, 1901. On this date four bids were received, and that of the Buffalo Dredging Company, of Buffalo, N. Y., being the lowest, the work was let to this firm on April 10th, 1901.

The type of structure adopted, Fig. 25, consists of three main parts, *viz.*:

1.—Semi-submerged large concrete blocks, serving as a foundation and resting upon the stone-filled timber cribs, with their bases 2 ft. below mean lake level. The lake- and harbor- face blocks are provided on their abutting faces with joggle channels, and, in addition, these and all other blocks have sunken-panel recesses on their top faces, 6 ins. deep. The blocks are made in the open, and placed in position afterward.

2.—Concrete *in situ*, comprising the banquette deck, parapet walls, and parapet deck, founded on the concrete blocks.

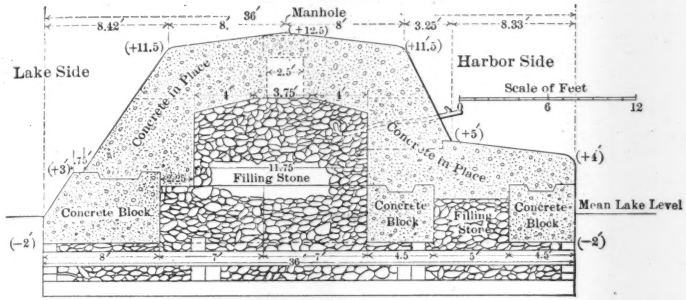
3.—The rubble filling.

The superstructure consists of isolated sections, each practically 36 ft. long, which linear dimension was determined by making five sections for each 180-ft. crib.

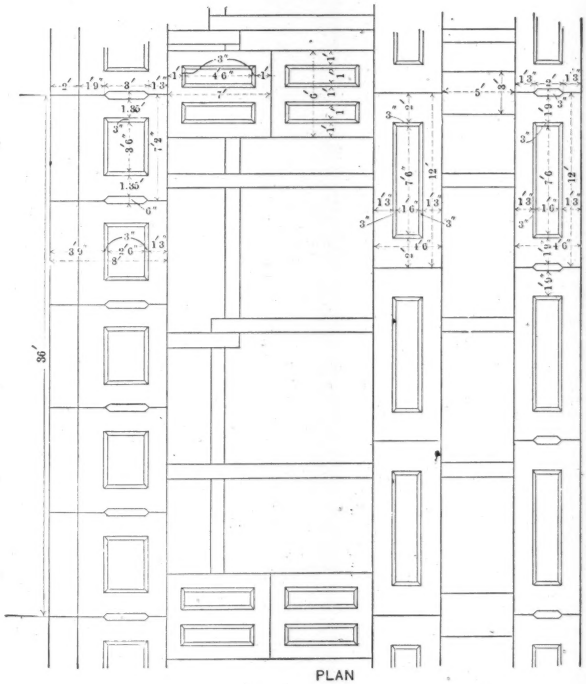
Each section is made up of: Five lake-face blocks, trapezoidal in section, 7.2 x 8 x 5 ft., each containing 9.45 cu. yds., and weighing, approximately, 18.9 tons; three harbor-face blocks, rectangular in section, 12 x 4.5 x 4 ft., containing 7.64 cu. yds., with an estimated weight of 15.28 tons; three interior-wall blocks, of the same dimensions as the harbor-side blocks, but without joggle channels at the ends, making their volume 7.75 cu. yds. and their weight 15.5 tons.

These blocks are laid in three parallel rows, the space between the lake-side and interior-wall blocks is 14 ft. wide, that between the harbor-side and interior-wall blocks is nominally 4 ft., but, generally, slightly more. In addition, there are two parapet cross-wall blocks, each 7 x 6 x 4 ft., containing 6 cu. yds., and weighing 12 tons. These blocks connect the lake-face and interior-wall blocks; they are spaced

CONCRETE SUPERSTRUCTURE
OF THE
SOUTH HARBOR BREAKWATER,
1901-1902



CROSS-SECTION



PLAN

FIG. 25.

at 36-ft. centers, and form the foundation for every two adjacent cross-walls. There are also banquette cross-wall blocks, 4 x 3 x 4 ft., also spaced at 36-ft. centers.

The blocks rest partly on the crib timbers, that is, the face walls, and on the ties where they occur, but most of their area is on the rubble filling.

The joggle channels, of the abutting faces of the lake-face and harbor-face blocks, were filled with concrete after the blocks had been placed in juxtaposition, thus tying them together and preventing any lateral movement. The enclosure formed by the blocks was filled with rubble.

These blocks served as a foundation for the banquette deck on the harbor side, and for the parapet walls on the lake side, all built of concrete *in situ*.

After the parapet walls had been formed, the enclosed space was filled with rubble, the parapet deck resting partly on the walls and partly on this stone filling.

The theoretical quantities contained in each 36-ft. section are as follows:

Blocks.

Lake-face blocks	5	at 9.68	cu. yds.	48.40 cu. yds.
Harbor-face blocks	3	" 7.75	" "	23.25 " "
Interior-wall blocks	3	" 7.75	" "	23.25 " "
Parapet cross-wall blocks	2	" 6.00	" "	12.00 " "
Banquette cross-wall block	1	" 2.22	" "	2.22 " "
Total blocks	14			109.12 " "

Concrete in place.

Banquette deck	54.55 cu. yds.
Parapet walls	81.04 " "
Parapet deck	99.19 " "
Total	234.78 " "

All the blocks were made on a construction dock, erected especially for this purpose near the breakwater site. It consisted of ten parallel rows of piles, closely driven, and capped by 12 x 12-in. timbers, on top of which was laid a 2-in. planed plank floor 32 ft. wide and 277 ft. long.

The mixer, hoisting engine and derrick were in the central part of the dock. Running the entire length of the same were two 20-in. elevated railroad tracks. The moulds in which the blocks were cast

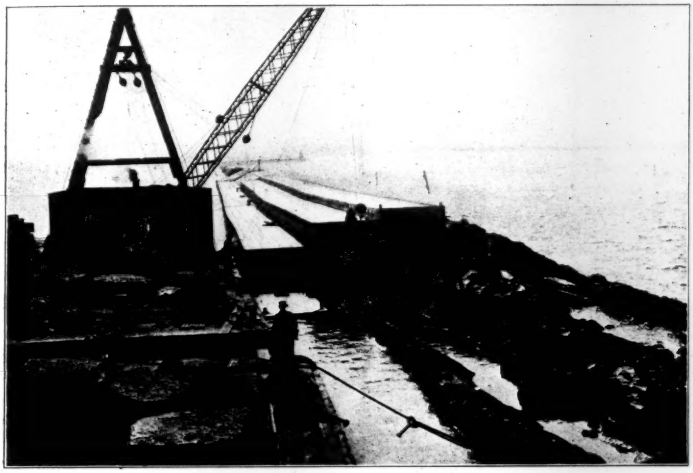


FIG. 1.—JUNCTION OF STONE BREAKWATER WITH SOUTH HARBOR TIMBER SECTION. 1902.

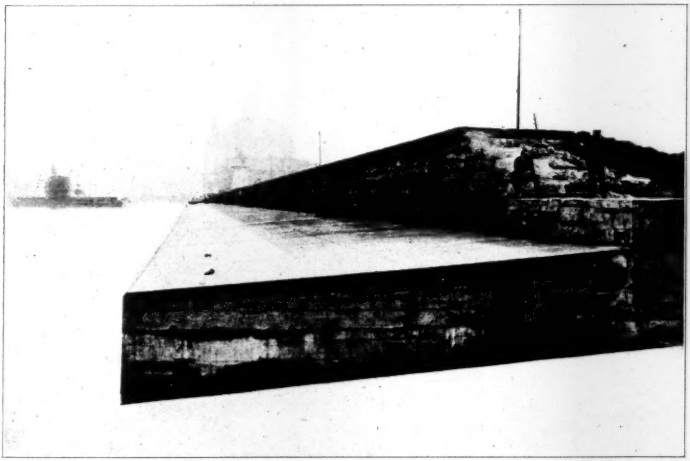


FIG. 2.—SOUTH PIER. ENTRANCE TO BUFFALO RIVER. STONE MOLE, BUILT 1826–1839. CONCRETE BANQUETTE. 1901.

were in four rows on either side of the tracks. There were, in all, seventy-five moulds, thirty for the lake-face, thirty-four for the harbor-face and interior-wall, and eleven for the parapet cross-wall, blocks. The banquette cross-wall blocks were made in place on the breakwater. The moulds had removable sides and ends, adjusted so as to be taken apart readily, which was done as soon as the blocks had become sufficiently hard.

A 4-ft. cubical mixer of the ordinary type was used. The broken stone was brought to the work in canal-boats, and for the transportation of the cement, sand and gravel, a remodeled dump scow was used, the pockets of which were utilized for storage purposes.

The composition of the concrete was as follows:

Lehigh Portland cement	1 part.
Screened gravel	1 "
Sand grit	2 parts.
Unscreened broken limestone	4 "

The unit of measurement was a bag of cement, which was considered to contain 0.9 cu. ft.

The concrete was made in batches containing four bags of cement. The gravel and sand grit were measured in wheel-barrows containing the equivalent of four bags of cement, or 3.6 cu. ft. The broken stone was measured in steel buckets, holding 21.6 cu. ft., in which the required quantity, viz., 14.4 cu. ft., was indicated by a mark. These steel buckets had been made for use with the 5-ft. cubical mixer in which was made the concrete to be placed *in situ*. In that work the batches contained six bags of cement, and, consequently, 21.6 cu. ft. of broken stone.

In making the smaller quantity, it would have been better had the bucket contained a false bottom, as it would have caused less strain on the inspector in trying to keep the stone down.

The concrete was made wet, from 2 to 2.5 cu. ft. of water being used in a batch. It was found that one batch of concrete made about 20 cu. ft. when rammed.

The characteristics of the various materials, as obtained from tests, were as follows: 1 cu. ft. of sand grit contained 0.47 cu. ft. of fine gravel, the grains of which ranged from $\frac{1}{8}$ to $\frac{1}{4}$ in., and 0.59 cu. ft. of sand, the respective weights being 47.2 and 59.3 lbs., equivalent to about 100 lbs. per cubic foot for the screened, or 106.5 lbs. for the combined, materials. One cubic foot of gravel contained 0.80 cu. ft. of gravel,

generally ⅜ in. in size, and 0.29 cu. ft. of sand, the respective weights being 81.5 and 29.2 lbs., again indicating closely 100 lbs. per cubic foot for the screened, or 110.7 lbs. for the combined materials. The broken stone showed 0.63 cu. ft. of coarse and 0.33 cu. ft. of fine stone and 0.11 cu. ft. of dust, weighing respectively 53.8, 30.4 and 11.0 lbs. for the screened, or 95.2 lbs. for the combined materials. This latter weight, however, is not correct for the broken stone as used in the work, because, in a box containing only 1 cu. ft., it is extremely difficult to shake down the stone as can be done in a larger receptacle. The actual weight of the unscreened broken stone was 101 lbs. per cubic foot. In these experiments the sand and gravel were sifted through a sieve containing 36 meshes per square inch. A ¾-in. sieve was first used for the broken stone, then the No. 36 sieve.

The concrete blocks were lifted by a large floating derrick, placed on deck scows, and carried to the breakwater. They were placed in the work by the same derrick. The plant for making the concrete *in situ* was installed in an old dismantled schooner, 137.3 ft. long, 26.6 ft. beam, and 11.6 ft. deep.

A 5-ft. cubical mixer was used. There were two hoisting derricks, one for lifting the ingredients from the scows to the mixer, the other to put the concrete in the work. The composition of the concrete *in situ* was the same as that of the concrete blocks, except that it was mixed in larger batches, the unit being six bags of cement, which produced 43.2 cu. ft. in each batch.

The concrete was made wet, about 3½ cu. ft. of water being used in each batch. About 28.5 cu. ft. of rammed concrete was made from each batch.

Three sets of forms were used for moulding the concrete *in situ;* one for the banquette deck, one for the parapet walls, and one for the parapet. The sheeting was of 3-in. stuff, spiked to a heavy framework. They were constructed so as to be taken apart readily and used repeatedly as long as they remained in proper condition.

The following is a *résumé* of the work of construction:

June 18th, 1901.	Commenced making concrete blocks.
July 10th, 1901.	First blocks set.
July 26th, 1901.	First mass concrete placed.
Nov. 18th, 1901.	Work for the season suspended.
April 14th, 1902.	Work for the season of 1902 resumed.
May 18th, 1902.	Work completed.

The cost of the concrete superstructure was as follows:

Old superstructure removed,	1 800.48	ft.,	at $12.00	$21 605.76
New timber	13 631	" B. M.,	" 100.00	1 363.10
Stone filling dredged	1 468	cu. yds.,	" 1.00	1 468.00
New stone filling	4 642	tons,	" 0.80	3 713.60
Concrete blocks	5 458.22	cu. yds.,	" 10.00	54 582.20
Concrete in place	11 807.23	cu. yds.,	" 9.40	110 987.96
Manhole covers	50		" 5.00	250.00
Mooring rings	25		" 55.00	1 375.00
Total cost				$195 345.62

or $108.49 per linear foot.

TABLE No. 23.—Cost of Extension of Buffalo Breakwater to Stony Point, up to December 15th, 1902.

Stone Breakwater.

Gravel, 245 242 cu. yds. at 13 cents	= $31 881.46		
Rubble	667 799.2 tons,	= 527 282.57	
Capping stone	284 878.5 tons.	= 348 975.66	
			$908 139.69

(7 250 lin. ft. $125.26 per linear foot.)

Stony Point Breakwater.

Dredging, 206 861 cu. yds. at 18 cents	= $37 234.98		
Filling, 193 383 cu. yds. at 13 cents	= 25 139.79		
Foundation stone	19 170.8 tons,	= 15 018.49	
*Timber cribwork	2 826.9 lin. ft.	= 200 031.86	
			$277 425.12

(2 802.9 lin. ft. $98.97 per linear foot.)

South Harbor Breakwater.

Dredging, 357 461 cu. yds. at 18 cents	= $64 342.98		
Filling, 321 695 cu. yds. at 13 cents	= 41 820.35		
Foundation stone	126 166.1 tons,	= 95 594.01	
Timber cribwork	2 763 lin. ft.	= 255 310.12	
			$457 067.46

(2 739 lin. ft. $166.87 per linear foot.)

	$1 642 632.29
Concrete superstructure (1 800.48 lin. ft.)	195 345.62
Total cost to date, exclusive of superintendence, office and other expenses	$1 837 977.91

*Includes $3 819.46 for repairs, on account of storms.

The North Breakwater.

This work comprises a breakwater, 2 203.4 ft. long, which covers and protects the entrance to the Erie Basin and Black Rock Harbor, between the Erie Basin breakwater and the Bird Island pier.

The substructure is timber cribwork, 1 101.9 ft. of which is 36 ft. wide, the remaining 1 101.5 ft. being 24 ft. wide. The depths vary from 22 ft. at the south end to 5 ft. at the north end. See Fig. 26.

On the axis of the structure the depths to rock ranged from 30 ft. to a little more than 5 ft., the stratum over the rock being gravel, sand, mud, and a clay of varying degrees of consistency. As the material overlying the rock did not furnish a stable foundation for cribwork, it was all dredged out and the deeper parts filled with rubble to an elevation of 22 ft. below mean lake level. In the shallower parts the cribs are fitted to the rock, as nearly as possible, and the irregularities corrected with rubble deposited where needed to level up the rock surface. On a foundation of this kind it was possible to construct at once a permanent concrete superstructure, no settlement being apprehended.

The superstructure is a shell of concrete from 3 to 5¾ ft. thick, filled with stone, founded on semi-submerged concrete blocks, resting on the cribs, the tops of which are sunk to 2½ ft. below mean lake level.

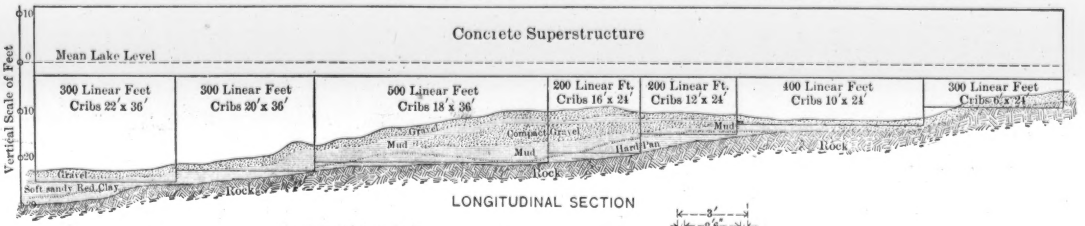
The structure is similar to that built on the ice-breaker section of the old breakwater and that of the South Harbor section.

As on other works forming Buffalo Harbor, unfavorable weather conditions hindered progress, and in two storms much damage was done to the uncompleted work. During the first, which occurred on September 11th and 12th, 1900, and is usually called the Galveston storm, the velocity of the wind was 80 miles an hour, and the water reached a level of 5.6 ft. above mean lake level. All the lake- and harbor- face blocks not covered by concrete were displaced, and one hundred and eighty-eight blocks were washed off the cribs. More than 1 600 cu. yds. of stone filling were also washed out. The storm of November 21st, 1900, also did more or less damage.

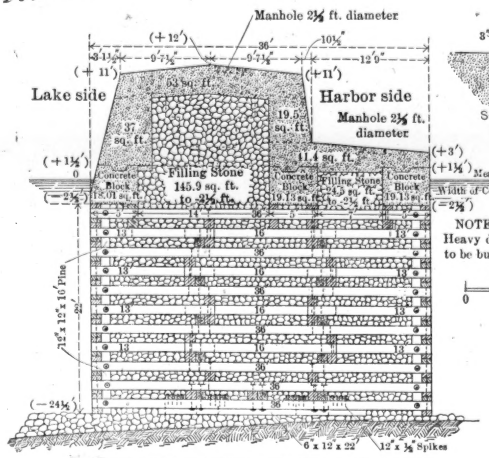
The work was done under the River and Harbor Bill of March 3d, 1899.

The contractor was J. B. Donnelly, of Buffalo, N. Y. The work was started in August, 1899, and completed on June 4th, 1901.

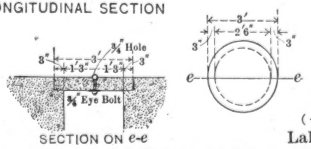
BREAKWATER TO PROTECT BUFFALO ENTRANCE TO BLACK ROCK HARBOR AND ERIE BASIN.

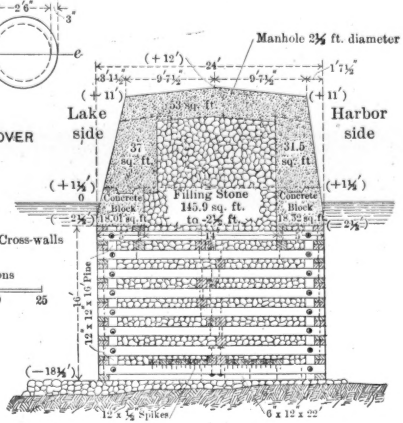


LONGITUDINAL SECTION



CROSS SECTION WITH DEEPEST 36-FOOT CRIB





CROSS SECTION WITH DEEPEST 24-FOOT CRIB.

FIG. 26.

TABLE No. 24. TOTAL COST OF THE NORTH BREAKWATER.

Trench excavation	41 657	cu. yds.	at	$0.20	$8 331.40
Rip-rap	7 133	tons	"	0.84	5 991.72
Foundation stone	9 325	"	"	0.69	6 434.25
Cribs, 22 x 36 ft	300	lin. ft.	"	62.00	18 600.00
" 20 x 36 ft	300	" "	"	56.00	16 800.00
" 18 x 36 ft	500	" "	"	48.00	24 000.00
" 16 x 24 ft	200	" "	"	30.00	6 000.00
" 12 x 24 ft	200	" "	"	24.00	4 800.00
" 10 x 24 ft	400	" "	"	21.00	8 400.00
" 6 x 24 ft	300	" "	"	15.00	4 500.00
Filling stone	13 640.9	tons	"	0.69	9 412.21
Concrete blocks	4 613.75	cu. yds.	"	7.10	32 757.63
Mass concrete	12 320.9	" "	"	5.65	69 613.08
Manhole covers	70		"	3.00	210.00
Mooring rings	12		"	50.00	600.00
Anchor bolts and plates	3 774 lbs.		"	0.10	377.40
Total					$216 827.69
Cost of recovering and replacing concrete blocks and stone filling washed from the breakwater by storms					6 000.00
					$222 827.69
Engineering and office expenses					13 450.41
Total					$236 278.10

CANADIAN QUARRY.

The quarry from which was obtained much of the rubble, and all of the capping stone, used in the extension to Stony Point, is located at Windmill Point, Ontario, Canada, on the north shore of Lake Erie, about 5 miles west from Buffalo.

The tract of land upon which the quarry is located contains about 80 acres, bordering on and lying south of the Grand Trunk Railway, having a frontage of about ¼ mile on the railway, and a depth, toward Lake Erie, of about ½ mile. The land does not extend to the water front, and, in order to reach the latter, the contractors obtained in the beginning a right of way, 20 ft. wide, over the intervening property, which, later, they acquired by purchase.

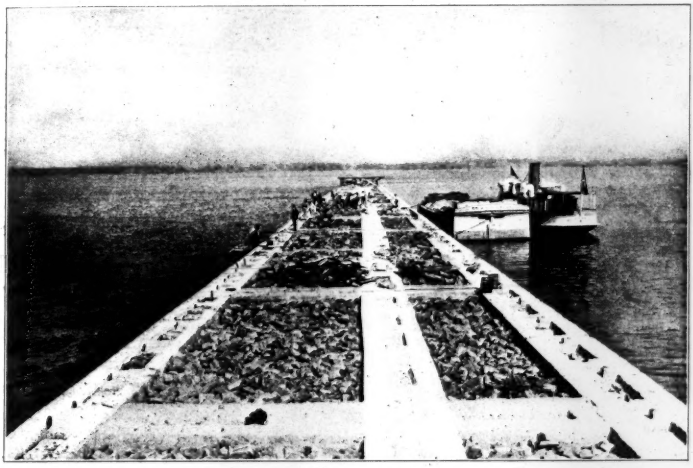


FIG. 1.—NORTH BREAKWATER. CONCRETE BLOCKS. FOUNDATION FOR MASS CONCRETE IN PLACE. 1900.

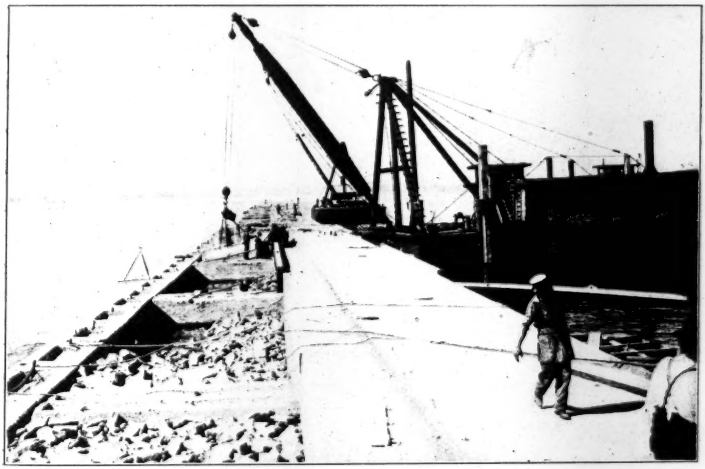


FIG. 2.—NORTH BREAKWATER. BANQUETTE MOULDED. SETTING LAKE-SIDE CONCRETE BLOCKS. 1900.

The land is approximately level, and the stone was quarried from below the surface, the overlying earth being from 2 to 5 ft. in depth. The quarry was opened by removing this earth for a length of about 900 ft. and a width of about 50 ft. As originally laid out, the direction of the stripping was approximately east and west, but, after about 600 ft. of the rock had been uncovered, a pocket of earth, from 6 to 7 ft. in depth was encountered, so the direction was changed somewhat. The shorter section is 300 ft. long, and the angle between the sections is about 30 degrees.

After completing the stripping, tracks were laid along the south edge, and a line of derricks erected between the tracks and the stripping. The rock was then blasted out by heavy charges of dynamite, ranging in power from 30 to 50 per cent. The result of these heavy charges was the breaking up of the rock into very small fragments, making much of the material useless for breakwater work. Much of it, however, was used by the contractors for filling the cribs of their loading dock, ballasting track and other purposes.

Soon after commencing blasting, it was found that the strata at the eastern end of the quarry were thicker than those at the western end, the layers running from 2 to 5½ ft. in thickness. Blasting operations, therefore, were confined to the western end, the excavated material being used for rubble, while that at the eastern end was reserved for revetment and capping stone.

For the transportation of the stone, a 3-ft. narrow-gauge railway, with the necessary sidings and turnouts, was laid from the quarry to the loading dock, about a mile distant.

The approach to the dock is about 835 ft. long, and the dock, as originally built, consisted of a series of isolated timber cribs connected by stringers. The cribs are of hemlock, and are 20 ft. wide and 22 ft. long, their depths varying with the depth of the water, which, at the outer end, is about 14 ft. The cribs are filled with stone and are wide enough for two tracks. The spans between the cribs are 20 ft., and the track stringers, of which there are two under each rail, are 24 ft. long, 8 ins. wide, and 12 ins. deep.

Since the cribs were sunk, about 400 ft. of the approach has been filled in with débris from the quarry.

The loading dock proper, at the outer end, is about 365 ft. long, and consists of cribs placed end to end, with heavy rip-rap on the

west or seaward side. At the extreme end there is also an L-crib, serving as a protection.

There are two engine-houses, on separate cribs, and two loading derricks on the edge of the dock.

Originally, both flat cars and dump cars were used for carrying the quarried material. The flat cars were intended for carrying capping stone, and also for skips and boxes, which were loaded on the ground in the quarry, then hoisted to the flat cars by derricks, then transported to the loading dock and there unloaded by the derrick. When the dump cars were used, the material was loaded into skips by hand, and these were then unloaded into the dump cars by derricks, or, where the stones were of large size, they were loaded directly by the derricks.

In the beginning (the fall of 1897), the plant at the dock consisted of a stiff-leg derrick, operated by a hoisting engine, and two stationary loading chutes. In unloading rubble from dump cars, it was dumped into the chutes, from which it was allowed to run into the pockets of the scows. There was much difficulty in using these loading chutes, owing to the fluctuations in the level of the lake, the scows often jamming under them and causing more or less damage. The dump cars, also, were not especially adapted for carrying rubble, and it was soon evident that the loading and dumping of the skips was unnecessary labor, when they might as well be carried at once on a flat car and unloaded at the dock by derricks. The change was adopted, although it put twenty-five cars out of service.

In opening the quarry, a depth of 30 ft. below the surface of the ground was contemplated, but this depth was only reached at a few points, at the end of the working season of 1897. Soon after the opening of the working season of 1898, the quarry reached a depth of 30 ft. for a length of about 600 ft. A double track was then laid along the south face of the quarry at a height of 6 ft. above the bottom, the foundation of the tracks being the débris from the quarry operations. The derricks, which stood formerly at the top of the quarry, were removed to the bottom. An inclined plane was also built and used for drawing the loaded cars out of the quarry.

The rock quarried is a stratified limestone, dark gray in color, hard and brittle in texture, and weighs about 166 lbs. per cubic foot. An analysis of the rock is given in Table No. 26. The courses dip to the east and get thicker as they are worked to the

north. The thickness of the different courses varies considerably in the length of the quarry. The stone is taken out in four ledges. At the west end the first ledge is about 10 ft. thick, and the first 150 ft. or so is shelly and only suitable for rubble, very few capping stone being quarried from it; in fact, very few capping stone are obtained from any of the courses in the first 150 ft. At the middle of the north side, or near the angle, the first ledge is about 12 ft. thick, and has a 3-ft. shell on top, which drops off at 150 ft. east of the angle. This shell is used principally for rubble. At the east end the first ledge is 8 ft. thick, with a 30-in. shell on top. The second ledge varies from 7 ft. at the west end to 10 ft. at the east end. The third ledge is more uniform, and is about 5 ft. thick. Under this is what is called the flint course. It is from 6 to 7 ft. thick at the west end, but at the east end little of it was quarried, as the grade of the bottom of the quarry rises for the incline located there.

The strata on the south side of the quarry are very much laminated, and, consequently, stone from that portion was only used for rubble. This class of material was quarried by blasting holes charged with dynamite, producing generally large, irregular-shaped fragments, which, when required, were reduced in size by "mud capping," which consists of placing two or three sticks of dynamite on a stone, covering with mud and firing.

When it was decided to make part of the breakwater extension to Stony Point a rubble mound, the question of procuring the large revetment and capping stone was paramount. The original estimate called for 220 000 tons of stone of this class, to be in blocks quite symmetrical in shape. This quantity of stone was equivalent to about 32 000 blocks, and, in place, would occupy an area approximately 300 ft. square and 30 ft. deep. It is extremely doubtful if a quarry which could furnish this quantity, within a limited time and at reasonable cost, could have been found in the immediate vicinity of Buffalo. The acquisition of the Canadian quarry, however, solved this problem successfully, and it can be safely said that there are few quarries equal to it, as far as concerns special adaptation to furnishing the required stone. These advantages consist mainly of three important items, namely, an excellent quality of stone, the thickness or depth of the ledges, and their normal or horizontal stratification.

In quarrying the large blocks of stone, the so-called Knox system

of quarrying was used. This system is the invention of Mr. John L. L. Knox (deceased), of Allegheny, Pa., and the main patent (No. 291 606) was granted on January 8th, 1884. This and other patents have expired, and have been open to free use since July 1st, 1902. The essential features of the Knox system consist in controlling and directing the force produced by the explosive, so as to make it break the rock in a prescribed line without injury to the standing rock. This is done by a blast hole of a peculiar shape, and a special method of tamping and charging it.

The shaping of the hole, the special tool to make it, the methods of charging and tamping, together with their combination, represent the special features of the system.

The practical methods of quarrying by this system varied somewhat in the different seasons. In the beginning, the blocks were quarried in irregular sizes. This was soon modified by making one dimension always 6 ft., this being the contract requirement for the depth of the "giant pavement," as it was termed in the specification. After the quarry had attained a depth of 30 ft., an approximately plane face had also been secured. A line of holes, 6 ft. from this face, was then drilled, loaded and exploded, producing a long block of stone of this width and of varying depth, depending on the thickness of the ledge. These blocks were then cut into parallelopipedons, or cubical blocks. Later, the line of holes was drilled 12 ft. from the face.

The final practice, however, was as follows: The overlying earth was stripped off the rock for a width of 20 ft. or more. The face of the quarry was then worked in four stages. To work the first or top ledge a line of holes 18 ft. from the face was drilled. These holes averaged 4 ft. from center to center, and extended to within 6 ins. of the bottom of the ledge, in this case about 12 ft. deep. The number of holes for one shot varied from fifty to sixty-five, although many shots containing only from twenty-five to thirty holes were made. Where possible, the holes were made to start and end at cross-seams, so that the ledge could be moved out without being broken up.

Sargeant machine drills, operated by steam, were used. In starting a hole a $2\frac{3}{8}$-in. drill was used, making a slightly tapering hole, 2 to 3 ft. deep, the drill then requiring resharpening. The second drill was from $\frac{1}{8}$ to $\frac{1}{4}$ in. smaller than the first, and drilled about the

same depth, when a still smaller one was used, and so on, until the full depth was attained, the diameter of the last drill being generally $1\frac{5}{8}$ to $1\frac{3}{4}$ ins.

When the holes had reached the required depth, a reamer was used The face outline of this tool is in the form of a hexagon. Referring to Fig. 27, taken from the original patent specification, the sides, *c c*, are parallel, and are equal in length to the sides of a square inscribed within the circle of the cylindrical bore. This brings each of the corners, *d*, in contact with the surface of the bore, and thus guides the tool and insures the passage of the plane bisecting the grooves through the center or axis of the bore.

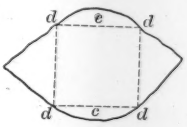


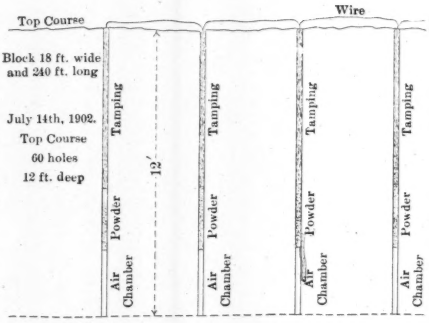
FIG. 27.

In practice, however, the central part of the reamer was made oblong instead of square, the width being generally 1 in. for all sizes used, and the long diameter being variable, depending on the size of the hole in which it was to be used. From Fig. 27 it is very evident that the object of the reamer is to cut two angular grooves lengthwise of the drilled hole. These grooves are located in the axis of the line of drill holes.

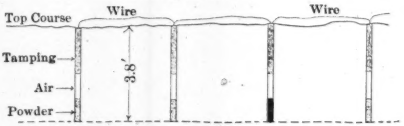
At first, there was more or less difficulty in guiding the reamers so that the grooves would be in proper position, or perpendicular. The square shank or stem was gripped by tongs or nippers, but, in driving, the tendency of the shank was to turn, thus making a twisted groove.

To obviate this, castings, about 4 to 6 ins. long, and fitting the top of the hole, were made. In these there was a square hole, in which the shank fitted. They also had two opposite projecting lips to fit into the groove, the idea being to hold the reamer firmly and keep it from twisting. It was found afterward that these lips were not needed, and they were discarded. The reamers were driven both by hand and by steam. In charging the holes two methods were used, as shown in Fig. 28. In one the powder was placed at the bottom of the drill hole, a wad of grass was then forced down to within 2 to 4 ft. of the powder, and then the tamping was put in. This left an air space or chamber above the explosive. In the other method the wad of grass was put in first, keeping it about 3 to 4 ft. above the bottom of the hole, then putting in the powder, and then the tamping. In this case the air space or chamber was below the explosive.

The factor determining which method of charging to use depends greatly on the tightness of the bed, that is to say, whether the seam separating the ledges is or is not well defined, or, in quarry parlance, whether or not the bed is free. Clay tamping was generally used, and sand tamping seldom. The series of holes was fired simultaneously by an electric battery.



GENERAL METHOD OF CHARGING HOLES IN QUARRY



METHOD OF CHARGING USED OCCASIONALLY.

FIG. 28.

The details of a shot made on the evening of October 1st, 1900, may be of interest. The ledge was about 250 ft. long and 18 ft. wide, the west half was about 10½ ft. thick, and the east half 8½ ft. thick. The drill holes were of the usual tapering diameter, 2½ ins. wide at the top, and extended to within about 6 ins. of the bottom of the ledge. There were sixty-four charges, spaced about 3 ft. 10 ins. apart, and 150 lbs. of powder was used. The powder was placed in the bottom

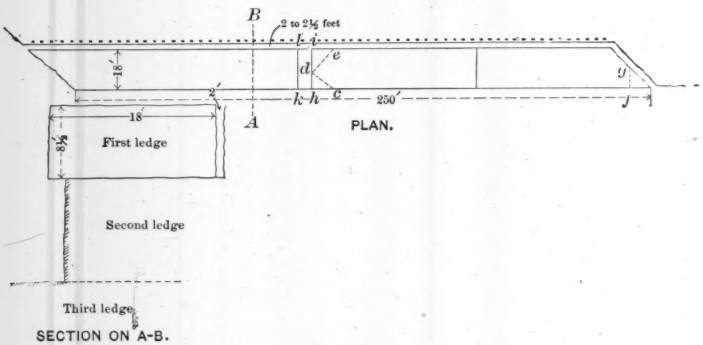
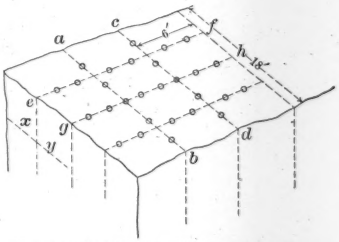


FIG. 29.

of each hole and then a wad of grass was put in, leaving an air space of about 3 ft. above; the remainder of each hole was tamped with clay.

The position of the ledge after the shot is shown in Fig. 29. The ledge was moved out bodily from 2 to 2½ ft. toward the quarry. Only a few cracks were made across the ledge. The weight of the stone moved was more than 3 500 tons.

Openings are made at the cracks, by first drilling along the lines *c–d*, *d–e*, *f–g*, and taking out angle stones and rubble until an opening is made. After the ledge is shot off it is cut into rectangular blocks, about 6 x 18 ft. (or 7 x 18 ft. for top angles), by drilling along the lines, *a–b*, *c–d*, etc. (see Fig. 30), 6 ft. apart, five holes being drilled in each line. Then each block is cut into four by drilling and blasting along the lines, *e–f*, *g–h*, etc. When necessary, each block is split by boring into the side, as at *x–y*, and splitting by wedges or light charge charging the holes along the lines, *a–b*, *c–d*, etc., about as much powder is used as can be held in the palms of two hands. The top ledge requires more powder than any of the others, as it is a softer limestone.



SKETCH SHOWING GENERAL METHOD OF CUTTING UP A LEDGE.

FIG. 30.

On October 24th, 1900, a block of stone 180 ft. long and 18 ft. wide with an average thickness of 9 ft., was blasted from the second ledge, at the eastern end of the north face of the quarry. A line of holes was drilled 18 ft. from the face of the top ledge, and about 3 ft. 6 ins. apart. The holes varied in diameter from 2½ to 1⅞ ins., and they were from 8 to 9 ft. deep. There were fifty-two holes in the line, and three kegs, or 75 lbs., of powder were used, the holes being fired simultaneously by electricity. The holes were tamped with sand instead of clay. Sand gives very good results and saves a great deal of time in loading, as it does not require as much tamping. The fifty-two holes were loaded in an hour, while it would have taken at least 2½ to 3 hours to load with clay. In case the charge fails to explode, it is much easier to clean out holes which have been tamped with sand. To clean out a

charge, a hose is attached to a small pipe and steam is blown into the hole; water is also allowed to run in, and thus blow out the charge. The block described weighed about 2 400 tons, and was moved 2 ft.

The best capping stones came from the first three ledges on the north side of the quarry, and toward the eastern end. Very little capping stone was obtained from the bottom or fourth ledge, as it was very flinty and hard to work. For this reason it was used mostly for rubble. All courses on the north side at the west end were much laminated, and therefore, were used principally for rubble.

Referring to Fig. 31, which is a cross-section of the upper part of the superstructure of the stone breakwater, it will be observed that the extreme top is composed of three separate stones, which are

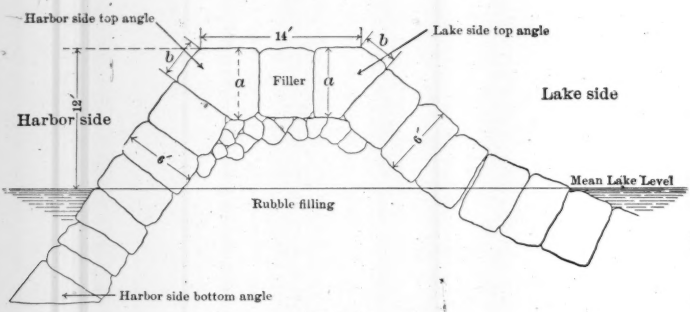


FIG. 31.

termed, respectively, the harbor-side top angle, the lake-side top angle and the filler. These angle stones, as well as the harbor-side bottom angles, were all quarried to the shape required by blasting, suitable templates being used to locate the drill holes.

Numerous fissures or cracks crossed the quarry face at an angle of about 45° and, in every instance, advantage was taken to utilize the cracks as one of the faces of the stone. In other cases the stones were quarried especially out of the whole. The depth of the stone, *a*, closely approximated 6 ft., while the dimension, *b*, varied from 1 to 3 ft.

About 2 200 stones of each kind, or 6 600 in all, were quarried. More than 40 000 stones of all kinds, for the revetment and capping, were used in the work.

As previously mentioned, Sargeant steam drills are used in the quarry. There are three sizes, large, medium and pony, designated as *F*, *C*, and A^{35}. Generally, six large, twenty-one medium and seven pony drills are in service.

The large drills are capable of drilling daily about 68 ft., the medium drills, 58 ft, and the pony drills, 22 ft. The latter were used mostly on the sides of large blocks which required splitting, or in facing up badly-shaped stones.

Steam for the drills is furnished by five horizontal fire-tube boilers of about 40 H.-P., and carrying 75 lbs. of steam. The boilers are about 18 ft. long and 4 ft. in diameter. The fire-box is about $4\frac{1}{2} \times 3\frac{1}{2} \times 3\frac{1}{3}$ ft. There are fifty-four 3-in. fire tubes, 11 to 12 ft. long. The heating surface is 546 sq. ft. The boilers were made by the Erie City Iron Works, of Erie, Pa., and by Farrar and Trefts, of Buffalo, N. Y. They are on the level ground about 100 ft. from the face of the quarry, the steam being carried to the drills in uncovered pipes, as the quarry is not worked during the winter.

There are sixteen derricks for handling the stone. They are of the ordinary mast and boom type, guyed by six lines of 1-in. galvanized-wire cable. The mast is 50 ft. high, with a 45-ft. boom, both of pine. The castings were made by the American Hoist and Derrick Company, of St. Paul, Minn. These derricks can handly safely a 20-ton block. The boom was usually kept in a fixed position, and was raised and lowered by hand.

There are eight hoisting engines, and, with one exception, they are of the usual skeleton type; that is, without boilers attached. There are seven double-drum engines with double cylinders, 8 x 12 ins., and are rated at 20 H.-P. All were made by M. Beatty and Sons, of Welland, Ontario. There is also one hoist, with boiler attachment, made by the American Hoist and Derrick Company, of St. Paul, Minn., having 7 x 10-in. cylinders. Each engine works two derricks, doing the hoisting only.

Steam for the skeleton engines is supplied by two boilers similar to those supplying the drills. A hoisting engine with boiler, and 8 x 12-in. cylinders is used for drawing the loaded cars up the inclined plane. From 8 to 12 loaded cars constitute a load.

Water for the boilers is taken from Lake Erie during warm weather, by a 6 x 4 x 6-in. Worthington pump at the loading dock,

and having a capacity of 75 galls. per minute. The water is pumped to an elevated tank at the quarry, through a 2-in. pipe laid beside the railroad.

In freezing weather the water from the lake is shut off and water is taken from the quarry pit, where it accumulates rapidly and is taken care of by two Worthington pumps: A 7½ x 7½ x 6-in., with a capacity of 300 galls. per minute; and a 7½ x 5 x 6-in., with a capacity of 100 galls. per minute. Steam for these pumps is furnished by a 25-H.-P. boiler made by Farrar and Trefts, of Buffalo, N. Y.

The loading plant at the dock consists of two large derricks, each operated by an 18-H.-P. double hoisting engine with 7 x 10-in. cylinders, with boiler attached. The derricks are swung by a 10-ft. horizontal bull wheel.

The rolling stock consists of two small locomotives and about one hundred flat cars, also a number of dump cars.

The beds of the flat cars are 10 ft. long, 5 ft. wide and are 2 ft. 9 ins. above the rail level. The side sills are 6 x 10-in., the end sills 6 x 9-in., framed with their tops 5 ins. above the side sills, a 4 x 5-in. filler being placed on top of the latter to even up. All the woodwork is white oak.

The wheels are of cast iron, 2 ft. in diameter. The cars were made by the Union Car Company, of Depew, N. Y.

The skips or boxes for carrying rubble are of 2-in. white oak. They are 7 ft. long on top, 5 ft. on the bottom, 5 ft. wide and 2 ft. deep, and each carries from 3 to 4 tons of stone.

Organization of the Force.—A superintendent has general supervision of the entire quarry. Immediately under him there is a walking boss whose main duty is the loading of the stone. Four foremen have charge of quarrying the capping stone and laying out work for the drillers, each having his own section of the quarry. There is also one foreman for each of the sixteen derricks, who has from six to eight men under him. The stripping gang consists of a foreman, ten laborers, and the necessary teams. A gang of three men in charge of a foreman attend to the track repairs. The blacksmith force consists of ten smiths and nine helpers, the main work being the sharpening of the drills. In addition, there are machinists, carpenters and other artisans. The total force averages about two hundred and seventy men.

The wages paid at the quarry were as follows:

Superintendent	$100.00	per month.
General foreman	85.00	" "
Machinists	70.00	" "
Time keeper	60.00	" "
Teams	0.30	per hour.
Locomotive engine drivers and blacksmiths	0.25	" "
Foremen	0.22½	" "
Drillers (on steam drills)	0.20	" "
Hoist enginemen, firemen, blacksmith's helpers, carpenters and watchmen	0.17	" "
Driller's helpers and laborers	0.15	" "
Water boys	0.10	" "

TABLE No. 25.—TOTAL OUTPUT OF THE CANADIAN QUARRY, BY YEARS; AND COST OF LABOR ONLY, PER TON OF STONE QUARRIED.

Season.	Rubble, in tons.	Capping stone, in tons.	Total stone, in tons.	Amount paid for labor.	Cents per ton for labor.
1897	27 501.5	383.4	27 884.9	.....	.....
1898	97 269.7	16 078.7	113 348.4	.....	.....
1899	111 557.4	42 425.1	153 982.5	$72 008.80	47
1900	122 946.0	54 988.7	177 934.7	72 576.83	41
1901	167 795.1	77 531.7	245 326.8	81 987.74	33
1902	137 279.2	93 470.9	230 750.1	83 519.37	36
Total	664 348.9	284 878.5	949 227.4	.....	.....

During the season of 1902 the following explosives were used:

Dynamite,	5 450 lbs., at 12 cents	.....	$654.00
Powder,	41 348 " " 5 "	.....	2 067.40
Fuses,	68 700 " 5 "	.....	3 435.00
			$6 156.40

The cost of explosives was about 2¾ cents per ton of stone quarried, 1 lb. of dynamite being used for every 42 tons of stone quarried, and 1 lb. of powder for every 5 tons of stone.

The coal consumption per day was as follows:

Boilers for drills,	5 at 1½ tons.....	7½ tons.
Boilers for derricks,	2 " 1½ "	3 "
Boiler for pumps,	1 " 1½ "	1½ "
Hoisting engine,	1 " ½ "	½ "
Hoisting engine,	2 " 1½ "	3 "
Locomotives,	2 " 1 "	2 "
Incline engine,	1 " 1½ "	1½ "
	Total.....	19 "

TABLE No. 26.—Analysis of Limestone from Quarry at Windmill Point, Ontario, used in Buffalo Breakwater.

Sample.	Moisture.	Silica.	Iron and alumina.	Calcium carbonate.	Magnesia.	Phosphoric acid.	Remarks.
No. 1.....	0.226	30.15	1.03	66.84	1.29	0.021	Top ledge.
2.....	0.29	30.34	1.27	65.05	1.30	0.025	Middle "
3.....	0.36	45.30	0.94	50.29	0.86	0.16	Bottom "
1c.....	0.20	21.33	0.99	72.52	1.39	0.009	Intermediate "

Conclusion.

Since the preparation of this paper, the breakwater extension to Stony Point has been completed, as far as the contract with Hughes Brothers and Bangs is concerned. The last stone in the rubble-mound or stone breakwater was laid, with simple ceremony, on September 24th, 1903, in the presence of Major Theo. A. Bingham, Corps of Engineers, U. S. Army, who succeeded Major Thomas W. Symons, on May 1st, 1903, in charge of the Buffalo District.

The entire work done during the season of 1903 was the reconstruction of the sunken part, in strict conformity with the stipulations, as outlined in another part of this paper.

Such capping stone as were readily recovered by means of a diver were taken out and reset. The number recovered was about 1 800, or about 11 000 tons.

For the greater part of the season there were three floating derricks on the repair work, one generally resetting the recovered capping stone, and one setting the new capping stone. The other derrick was engaged part of the time in handling the rubble hearting, and the

remainder of the time in placing rip-rap in front of the adjoining South Harbor Section. The cost of the reconstruction was as follows:

27 634.3 tons of rubble from Canada, at $0.80....	$22 107.44
15 338.2 " capping " " 1.25....	19 172.75
Force account, recovering and resetting capping stone.....	15 583.04
	$56 863.23
Cost to December 15th, 1902.....	908 139.76
Total.....	$965 002.99

The total cost of the work was $1 727 000.84, exclusive of office, inspection and other expenses.

The details of the various sections are shown in Table No. 27.

It is a matter of interest to note that the final cost is less than the original estimate, by which the bids were canvassed, which amount was $1 765 450.63.

TABLE No. 27.—Total Cost of Extension to Stony Point, Buffalo Breakwater Construction.

Hughes Brothers and Bangs, Contractors.

Contract dated January 27th, 28th, 1897.

Sand-Catch Pier:						
Piles.....	462		at	$11.00	$4 422.00	
Waling.....	1 139.35	lin. ft.,	"	0.25	284.88	
Tie-rods, screw bolts and straps.....	14 700.9	lbs.,	"	0.02½	367.52	
Stone filling.....	1 469.9	cu. yds.,	"	1.06	1 558.09	
						$6 632.44
Stone Breakwater:						
Gravel	245 242	cu. yds.,	at	$0.13	$31 881.46	
Rubble, U. S.....	203 391.4	tons,		.....	162 702.60	
" Canada.....	492 042.1	"		.....	386 687.47	
Capping stone, Canada....	300 216.7	"		.....	368 148.43	
Force account work.....					15 583.03	
						965 002.99
South Harbor Section:						
Trench excavation.....	357 461	cu. yds.,	at	$0.18	$64 342.98	
" filling.....	321 695	"	"	0.13	41 820.35	
Foundation stone.....	157 031.4	tons,	"	.....	117 654.49	
Timber cribwork.....	2 763	lin. ft.,	"	92.37	255 218.31	
Mooring cleats, extra timber, etc.....					2 723.62	
						481 759.75
Stony Point Section:						
Trench excavation.....	206 861	cu. yds.,	at	$0.18	$37 234.98	
" filling.....	193 383	"	"	0.13	25 139.79	
Foundation stone.....	19 170.8	tons.....			12 995.78	
Timber cribwork.....	2 826.9	lin. ft.....			197 065.15	
Mooring cleats, extra timber, etc.....					1 169.96	
						273 605.66
Total cost					.....	$1 727 000.84

DISCUSSION.

Mr. Pitts. THOMAS D. PITTS, ASSOC. M. AM. SOC. C. E. (by letter).—The writer draws attention to the following points:

In the first place, the capping stones of the stone breakwater appear to be quite regularly rectangular in form and of about 6 to 7 tons weight; and they are laid on the rubble mound without other bond than that given by the friction of one block against another. Records of experience abroad show that when a stone in such a pavement is removed from its place the whole pavement goes to pieces very rapidly. It seems to the writer that a comparatively small movement of either harbor- or lake-side angle stones might so disarrange the capping stones immediately above that they would be torn out by the waves, with the result that a large section of the pavement would be stripped from the core; and he would be much interested to know whether any indications of such a future mishap have been discovered. He would also like to know if it has been observed that the ice formation in winter has had any tendency to lift the capping stones from their beds.

In the second place, the sections of the breakwater (Fig. 22), showing the original section as designed and the section after settlement of the portion which failed, have almost exactly the same area above the lake bottom. This, to the writer's mind, indicates, not that the rear portion of the breakwater settled bodily into the soft clay bottom, but that it slid toward the harbor side, removing the support of the center section, which, of course, then settled down. The sliding was probably started by some slight settlement of the breakwater into the clay, but, from the drawings, it seems to the writer that it was probably more largely due to an insufficient mound or retaining wall of rubble at the toe of the rear slope, to too steep a slope of the harbor side of the breakwater, and to the use of gravel in the hearting. It seems likely that the latter was a very important factor in the settlement, because gravel, on account of its form, does not pack as solidly as rip-rap or rubble, especially when washed free from a portion of its matrix, as must have been the case with this gravel, dredged from the bed of the Niagara River. With regard to the portion of the breakwater first built, in which no settlement has taken place, its stability seems to be accounted for sufficiently by the more solid foundation which has prevented the initial settlement under wave action, which settlement would start the sliding; and it may be, too (it does not appear from the paper), that, under the part of the breakwater which sank, the top layer of clay is softer, and the coefficient of friction therefore much less than under the older part which has remained intact.

WILLIAM T. LYLE, ASSOC. M. AM. SOC. C. E. (by letter).—The history of Buffalo Harbor is incomplete without some description of the celebrated Black Rock controversy. The writer is in possession of letters, written by DeWitt Clinton, Myron Holley, David Thomas and Benjamin Wright, relative to this dispute; and these, supplemented by the reports of the Canal Commission and other official publications, throw considerable light on the subject in question. Mr. Lyle.

Along with the construction of the Erie Canal came a pressing need for a suitable port at the eastern end of Lake Erie. The choice of the terminus of the canal lay between two places; one, Buffalo, and the other, Black Rock. These places were about 4½ miles apart. There were five principal projects advanced. The three of least importance will be mentioned first. The dispute, in reality, lay between Projects 4 and 5.

1.—This plan provided for a harbor in the lake where the line of the main street of Buffalo intersected the beach. The harbor was to be formed by building a pier in a westerly direction and then extending it northerly parallel to the shore. The width of the harbor, from east to west, was to be 1 000 ft.

2.—It was proposed to excavate the neck of land near the ferry, and enter Buffalo Creek from the lake along the slip. This plan met with little favor.

3.—Another plan was to construct a large harbor at Black Rock by building a pier from Bird Island to Squaw Island, thence to Strawberry Island, thence to the head of Grand Island, and closing the opening between Grand Island and the eastern shore by a dam at White Oak Bluffs. This plan, by reason of its enormous cost and the likelihood of international difficulties, was soon abandoned.

4.—The fourth plan differed from the third in calling for the dam at the lower end of Squaw Island instead of at White Oak Bluffs. The north and south pier was to be about 500 ft. from the shore, and, at the upper end of Squaw Island, was to meet an embankment extending along the eastern shore of this island to the western end of the dam. This plan was worked out by James Geddes. In the plan adopted finally the pier was built nearer the main shore. A slight modification, and one which did not affect the embankment and dam already mentioned, was to turn the pier, at right angles with the shore, about 800 ft. north of Bird Island, thence to extend it two-thirds of the way across the east channel and thence at right angles down stream to the south end of the embankment. A pier built in this manner would obstruct the flow of the Niagara River greatly, and the modified plan was discarded.

The Black Rock project met with considerable favor, and was championed by General Peter B. Porter, a citizen of Black Rock, and his associates. They contended that in case of a storm, a harbor

Mr. Lyle. at Black Rock would be more easily entered than the one at Buffalo, which will be described later; that the canal would be shortened considerably by deciding in favor of Black Rock; that the harbor would be much larger than the one at Buffalo; and that hydraulic power of immense value would be developed. They maintained that Buffalo Creek brought down the sand that formed the bar at its mouth, and that Buffalo Creek was eminently unfit for a sheltering place.

The objection to the Black Rock plan was largely on account of its excessive cost. It was urged that in case of hostilities with the British, the harbor would be useless as long as they held possession of the opposite shore; and that in time of winter the floating ice hurried along in the swift current would demolish the pier. The prevailing winds would also interfere decidedly with the exit of vessels in attempting to reach Lake Erie. Another objection to the Black Rock Harbor arose from the fact that the bottom was rocky and afforded poor anchorage. It was also argued that the protection of Black Rock against the violence of the waves was due to the rapids in the Niagara River; and that this advantage would be partially lost should the harbor be constructed and the elevation of the water surface at Black Rock be raised to the surface elevation of Lake Erie. Since, in accordance with this plan, the canal was to be supplied with water from Black Rock Harbor, it was thought by many that it would be somewhat dangerous to stake the welfare of the canal on an artificial, and in some respects an experimental, construction. The work there would require much more time for its execution than that at Buffalo, and the time element was a very important one in view of the public clamor for the canal's early completion.

5.—A plan which did not thoroughly meet the approval of the Canal Commission, but which was endorsed by Governor Clinton and one of the commissioners, provided for the development of the mouth of Buffalo Creek as a receiving basin which would be both cheap and adequate. The plan was prepared by David Thomas, who, in 1820, was appointed Engineer of the western section of the Erie Canal, from the Genesee River to Lake Erie, and who was continued as Chief Engineer of the same section by a resolution of the Canal Commission passed in March, 1822. From his report, printed in the "Public Documents relating to the New York Canals," published in 1821, are copied the following extracts:

"The depth of water in Buffalo Creek is sufficient for a harbour. In taking soundings almost up to the ferry (which is 1 mile from the entrance) the least depth observed was 11 ft., and this only in two places; but the common depth up this stream is from 12 to 14 ft. About 50 rods above the mouth of Little Buffalo, we found 17 ft., and a few rods within that part of the entrance which is obstructed by sand, we found 19 ft. No injury to this channel need be apprehended from depositions of either sand or mud.

"The breadth of this creek, just above Little Buffalo, is full 16 rods; but 35 rods above, the breadth is only 12 rods; and the calculation of two triangles, taken further up the stream, at the respective distances of 40 and 70 rods, gave no material difference. Mr. Lyle.

"The shore is bold, and very little of the channel need be occupied by wharves.

"It appears that after freshets in this stream, the entrance is deepened from 12 to 15 ft.; but the common current is so sluggish as to permit the first gale to drive sand and gravel into it from the south. As the obstruction to navigation only originates from this simple cause, a remedy equally simple will be sufficient, and may be found in a pier or mole, as heretofore proposed by William Peacock. I think, however, that both its position and construction should be different.

"I would propose to commence it at the lighthouse, because there is no firm ground further north, and the work ought to be secure in the rear. Thus, if we began half way down to the point (which is only a loose bed of sand and gravel), Buffalo Creek might break through between the pier and the lighthouse. On the contrary, if we commence it above that building, the entrance of the creek will be less protected from northwesterly storms; and this view will be important whether we consider the entering of vessels or the drifting of sand into the channel. It appears that the direction of the gale shapes the gravel point, as its course on William Peacock's map varies considerably from its present position.

"The reason for locating the pier above the lighthouse was founded in misapprehension. There is no appearance of sand or gravel having ever been brought down by Buffalo Creek."

The foregoing report is dated October 15th, 1819.

It was Mr. Thomas' plan to commence the pier at the lighthouse and extend it North 70° West, 1 150 ft. The pier was to be made of timber cribs; the substructure was to be of round green logs filled with brush and gravel, and the superstructure of squared oak timbers loaded with large blocks of stone.

In a letter to Myron Holley, who was one of the Canal Commissioners, Mr. Thomas writes:

"If it should be deemed the best policy, notwithstanding, to expend the funds of the State in making a capacious harbour, and not to wait 50 years when it may be wanted, still Black Rock is not the place for such an expenditure. In truth, it is the most preposterous location that could be attempted. To make a harbour, worthy of being known as a national work, the site south of the Buffalo pier, originally selected by Joseph Ellicott, is unquestionably the best. At an expense not amounting to one-half of the Black Rock pier, a harbour may be formed in the lake which shall be twice as capacious as the anchoring ground to be enclosed by that work."

An argument in favor of Buffalo Harbor was its protection from storms and its freedom from ice. Mr. Thomas also urged that as the water of Buffalo Creek was much warmer in the early spring than the water in the Niagara River, canal navigation could be opened several weeks earlier if Buffalo Creek were made the terminus. This was afterward found to be the case.

Mr. Lyle. It may be of interest to consider Governor Clinton's opinions concerning the Black Rock project. In a letter to Mr. Thomas, dated July 25th, 1823, he writes:

"The Canal Board have adjourned to-day after affirming their former decision in favor of Black Rock. This has received my decided opposition, and is, in my opinion, totally indefensible."

Again, on June 15th, 1824, he writes:

"The decision in favor of Black Rock is universally condemned, and whatever the motives may have been, the deed itself will stamp the authors with indelible blame."

Again, on the 11th of July of the same year:

"With respect to the Black Rock Harbour (if artificially constructed) I never had but one opinion of its merits—that it will be a total failure unless at least a million dollars is expended on its construction. Time will verify your opinion, and there can be no doubt but that the men who have sanctioned this measure will repent of their folly."

The Black Rock Harbor was built, but it was decided to continue the canal as a branch to Buffalo. The Black Rock undertaking proved a failure. The pier suffered severely from the storms, and the harbor became choked with mud, as has been described in Major Symons' report. After the completion of the canal, the Village of Black Rock gradually declined and was soon overshadowed by its more fortunately situated rival.

Mr. Hughes. D. E. HUGHES, ESQ. (by letter).—This paper is very interesting and instructive.

The character of the clay, the cross-section of the mound, and the damage sustained in December, 1902, are matters of great importance for future reference, and there are a few points concerning them which the author has not yet made quite clear.

Fig. 22 shows a narrow berm on the harbor side, with a slope of 1 on 1½ below it, while Figs. 18 and 20 show a steeper slope, and practically no berm. The slope, 1 on 1½, is very near the natural slope of rubble in comparatively quiet water, and, taken in connection with the steeper slope above it, would give, in the absence of an ample berm, a combination which, though stable under vertical force alone, cannot have much reserve safety to withstand the resultant of gravity and the force of breakers, whether the latter be considered as acting on the structure as a whole or as a pressure transmitted through the water in the interstices to act on individual stones in the harbor slope.

It is noted that the drawings show that the gravel hearting was built to about its natural slope on the harbor side. A pile of rounded bodies like pebbles, when subjected to a load on top, tends to flatten out; hence, in this case, there would be some horizontal push from the gravel against the rubble armor, lessening its stability. Again, unless the interstices in the rubble be very small, the water in them

will be agitated to such an extent, when breakers are high on the lake side, that the gravel, unless very coarse, will seek a flatter slope by running into, and possibly through, the armor. Any such movement in the gravel would rob the superstructure of its support, and the capping stones, like a straining beam truss, or an arch, would produce horizontal thrust, and the weaker abutment—that on the harbor side—would be the one to yield. The paper does not state how coarse the rubble was, nor how fine the gravel. Mr. Hughes.

The author gives a quotation, leaving it uncertain whether or not he concurs in all its statements, in which it is said that "a length of about 1 000 ft. sank into the clay about 10 to 14 ft.," and Fig. 22 shows a typical section before and after the damage. The second section has as much area above the plane of the lake bed as is shown in the first, the loss at the top being fully compensated by the addition to the toe on the harbor side, thus proving, if the drawings be correct, that there was no sinking into the clay, and that the deformation was due to other causes.

Considering the height of the mound, the voids in the stone and gravel, the buoyancy of the water, and the distribution of the pressure over the wide base, the extra load on the foundation due to the structure is less than 1 ton per square foot. The author classified material dredged not far distant as moderately stiff clay, and walls of deep trenches stood much steeper than 1 on 1. It is not stated whether the steepness was limited by the character of the material or by the mode of dredging. In either case, such clay would be expected to bear a random stone breakwater 42 ft. high; hence it would be interesting to know what comparison was found, by borings or otherwise, between it and the clay along that part of the breakwater which suffered damage, also how easily the range piles drove along the same line.

Until the light of the final discussion is received, it will seem to the writer that the so-called settlement was a deformation due to some insufficiency in the cross-section, and not to any failure of the clay foundation.

G. H. Raymond,* Esq. (by letter).—The writer has read with pleasure this admirable contribution to the history of the Buffalo breakwater, and although he cannot discuss the paper from a technical standpoint, yet, having been much interested in the proposed 1 000-ton barge canal proposition for some years, he has been especially impressed with the magnitude of the Buffalo breakwater and its bearing on the commercial future of the State of New York and the nation at large. Mr. Raymond.

It is a strange fact that the building of this breakwater has been the greatest factor in bringing about the ratification of the canal referendum at the last election.

* Secretary, Canal Enlargement Committee.

Mr. Raymond. When the history of this movement for canal improvement is written, it will be made clear that the engineering ability of Major Thomas W. Symons, U. S. Army, in designing the breakwater, and the care exercised by Assistant Engineers Quintus and Low in carrying out the plans, have been of the greatest value to the canal proposition.

The value of the breakwater as a harbor protection was evident, but the problem of canal improvement needed some great impetus to impress its value on the people of the State.

The possible future of the eastern end of Lake Erie, as a point for making iron and steel, was recognized, but the harbor conditions at Buffalo were such that it was practically impossible to secure a sufficient area of land, adapted to an enormous steel plant and also having deep-water facilities for the ever-increasing capacity of lake craft.

The practical completion of the breakwater was in sight when a body of men, very largely interested in the iron and steel business, was looking for a site on the shore of Lake Erie where adequate rail connections and suitable deep-water facilities could be secured. They were also looking naturally for a cheap water route, for their products to the seaboard, and for eastern and foreign ores to be moved to their plant.

The deep-water facilities to be furnished by the breakwater, and the splendid railroad accommodations at Buffalo, coupled with the possible cheap water rate for foreign ores and those from Lake Champlain, together with the cheapest possible rate to the seaboard on their output, based on the active agitation for the 1 000-ton barge canal, induced these men to locate at Stony Point, inside the breakwater.

This enormous plant, wherein is being invested about $40 000 000, furnished the one great object lesson to show the people of the State what a magnet cheap transportation offered to the future iron and steel business of this country, and that the 1 000-ton barge canal was needed to round out the State's manufacturing possibilities.

There are numberless arguments in favor of the barge canal, and its ratification at the polls, by the largest majority ever given to any similar proposition in the history of the State, is evidence of the thorough appreciation of its future value; but one of the strongest arguments was the manufacturing possibilities along its length, through the centering of iron and steel interests along the Niagara Frontier.

The Buffalo breakwater alone made it possible to show the people that the iron and steel business would come to them if given proper encouragement, and to its building may very largely be ascribed the final success of the canal.

The Buffalo breakwater and the 1 000-ton barge canal are the two greatest propositions, in their respective lines, in the world, and it is a remarkable coincidence that they should be so intimately associated in location and in their effect on the future commercial history and prosperity of the State of New York.

Mr. Raymond. The completion of the Buffalo breakwater, the building of the 1 000-ton barge canal and the construction of a ship canal around the rapids of the Niagara River, which latter project is now being actively urged by Major Theodore A. Bingham, will insure forever the commercial supremacy of the State of New York and bring untold benefit to the entire territory tributary to the Great Lakes.

Dr. Fell. GEORGE E. FELL, M. D. (by letter).—The writer was engaged on the improvement of Buffalo Harbor, as U. S. Assistant Engineer, under Captain F. Harwood, Corps of Engineers, U. S. A., and presents the following historical notes in relation to the early stages of the work.

The construction of this great system of breakwaters was commenced in the spring of 1869. The construction under water consisted of 34 x 50-ft. cribs, of 12 x 12-in. hemlock. The lower courses, up to within 3 ft. of the water surface, were of hemlock. Above this level, the superstructure consisted of continuous 12 x 12-in. pine timbers, to a height of 8 ft. above the standard level of the water. This standard was established by the writer, under Captain Harwood's direction, by a fixed bench-mark on the lighthouse, 16.947 ft. above ordinary low-water level of the lake at the time. This level was adopted arbitrarily, and has been used in all subsequent observations. During the early years of the work the stage of the water was noted two or three times a day. These records, if in existence, should be of great value in determining the variation of the lake level.

The following notes, relative to the sinking of the first crib and other initial work, are taken from the writer's diary.

Preliminary arrangements for laying out the work were made on May 20th, 1869, and the lines of the south and north breakwaters were marked by buoys. On June 4th, the contractors had built Crib No. 1 to the full depth of twenty-three courses. On June 7th, at the site of Crib No. 1, six piles were driven, in the shape of the letter L, with the interior angle facing "up current" so that the crib would be held securely when floated into position.

The crib was towed to position during the night of June 8th, by moonlight. The crib was placed so that the proper corners fitted into the angles formed by the L-shaped group of piles and then sunk by filling it with stone. To align this crib, lanterns were placed on the line buoys.

Cribs Nos. 2 and 3 were built and placed in position in a similar manner. The placing of cribs was continued during the summer of 1869, although the season was exceptionally stormy, so that the work was prosecuted with difficulty, as will appear from the following instances:

On July 19th, Crib No. 5 was placed in line and properly secured. On July 21st, after about 200 cords of stone had been placed in the crib, a sudden southwest gale carried it away or broke it into two pieces, so that fifteen courses were carried ashore.

Dr. Fell. On September 29th, Crib No. 11, which had been sunk on September 27th, was carried ashore.

On November 18th, Crib No. 12 was washed ashore. Gales followed each other in quick succession, and disaster after disaster resulted, the short length of completed breakwater not affording sufficient lee for security in the work of construction. On October 23d, the last crib for the season was placed in line, making a length of 550 ft. of work for the season of 1869.

The season of 1870 was characterized by calm weather, a remarkable contrast to that of 1869. Crib after crib was placed in position, and the breakwater became at once a great boon to the lake mariner in affording a safe lee in stormy weather.

As to the foundation conditions, Mr. Low says:

"As the lake bottom of the area adjacent to the north end of the breakwater is generally rocky, overlaid with a thin layer of sand, no apprehension seems to have been felt in placing the cribs directly thereon; in fact, it is doubtful if any extended examination of the bottom, along the line of the proposed work, was made or considered necessary at the time."

As far as any extended examination of the lake bottom was concerned, this is true. Partial examinations, however, had been made by the writer, who began his work without previous knowledge or data to guide him.

The first attempts were made by the use of augers attached to gas pipes of varying lengths, the pipes being coupled, as usual, by screw sleeves. The pipes were prevented from turning at the joints by inserting ¼-in. iron pins into holes drilled through each pipe and sleeve. A common auger was first used, and found unserviceable; a "worm" auger was then tried, with more success, but what is termed a "pod" auger gave the best results. In carrying on the boring operations two small scows were anchored in position close to each other, and a connecting platform laid over the intervening space. The augers were operated from this platfrom. The greatest penetration obtained was about 8 ft. through very soft clay. In gravel it was with great difficulty that any penetration, beyond a few feet, could be made. The augers, which were from 2 to 3 ins. in diameter, were too weak to withstand the strain applied and broke continually. In several instances the pipes were twisted out of shape. The work progressed slowly in any material excepting very soft clay. Heavy weights were attached to the pipe in order to force the auger to penetrate the strata, but without appreciable success. The only means of judging of the character of the material was from the sound made by the pipe in boring. Observations made in this manner were found to be unreliable.

In January, 1872, the writer made more extended investigations, of the character of the bottom material, through the ice. For this purpose there was used extra heavy gas pipe with an outer diameter of 2

ins. and an inner diameter of 1 in. and in sections from 5 to 8 ft. long. These lengths were joined with ordinary sleeves. The lower or penetrating end of the pipe was furnished with a sharpened steel shoe, the inner diameter of which, for about ½ in. of length, was a trifle smaller than the internal diameter of the pipe. This shoe was attached to the pipe so as to prevent jamming of the threads of the screw in driving. A hardened driving head was also provided. Dr. Fell.

For the purpose of driving, pulling and handling the pipe, a hand-power pile-driver was constructed for use on the ice. It was provided with runners so as to be more easily moved. A horse was kept on hand to move it, an absolute necessity when the snow was heavy. A 175-lb. cast-iron hammer was used for driving. Operations with this apparatus were conducted with great success, and the bottom of the lake was examined to the rock bed. In all cases the inside of the pipe was partially filled with the material forming the lake bottom. After the pipe was raised the shoe was removed and the contents of the pipe forced out with a rod which closely fitted the inside of the pipe. In driving, the different velocities of the pipe in penetrating the various strata indicated their relative densities (or resistances), the depths being also noted. As the pipe, when withdrawn, contained a sample of the material, the character of each stratum was known.

Owing to the cold weather, the contents of the pipe froze quickly; so that it was necessary to provide a stove, protected by a tent, and to use hot water for thawing out the pipe. The power afforded by the windlass attached to the driver was sometimes insufficient to withdraw the pipe; when this was the case various combinations of 8-in. and 12-in. pulley blocks were used in connection with the windlass, which was manned by five men. This additional power was required in cases where the pipe had penetrated sand and gravel in varying proportions. The average depth of water at the site of the work was 27 ft. The greatest penetration of the pipe was 22 ft. through red clay to the solid rock. The greatest length of pipe filled with material was 20 ft. These borings showed that the foundation of the cribs to be sunk during the season of 1872 would be in soft clay, and that considerable settlement might be expected. Therefore, special precautions were taken to keep the cribs in line by piling heavily, and the superstructure was built up to the full height. Toward the close of the season of 1872, however, this foundation showed evidence of serious weakness, as will appear from the following occurrence.

On September 29th, 1872, a violent southwest gale set in, and increased in fury during the following night. Up to 6 P. M. on this day the work was intact, but on the morning of September 30th, it was discovered that the whole of the addition of the season of 1872 had been displaced and had settled to a considerable extent. Cribs Nos. 42 to 47 had settled so that the top of their superstructure,

Dr. Fell. instead of being 8 ft. above water, was in some places 2 ft. below water. The cribs had also worked out of line to a considerable extent. This disaster emphasized the necessity for a thorough examination of the bottom of the lake on the line of the proposed extension of the breakwater.

The following is a description of a plant used for examinations of the lake bed in the summer of 1873 on surveys for the Harbor of Refuge, at Cleveland, Ohio. It consisted of a self-propelling scow and driver, which had formerly been used for driving stakes or poles for fish nets. The scow was about 30 ft. long, 12 ft. wide and 2 ft. deep. At the stern was a central longitudinal open well, 10 ft. long and 1.5 ft. wide, in which a paddle wheel was placed. A small upright engine and boiler of about 6 H.-P. was used for propulsion and operating the boring apparatus. A 413-lb. hammer was used in driving. With this plant, borings were made in water from 24 to 37 ft. deep, and a penetration of 20 ft. made through shingle, quicksand and blue clay. In these depths it was found that there was considerable vibration of the pipe, indicating the desirability of using an outer pipe of larger diameter. The cost of operating this plant was $30.50 per day.

During the spring and summer of 1874 a survey of the harbor at Buffalo was made by the writer, under the direction of Major Harwood, for the Board of Engineer Officers of 1873–74. The plant used for ascertaining the character of the lake bed was designed by the writer as an improvement on that used at Cleveland. It was a self-propelling "steam rod driver," and consisted of a scow 40 ft. long, 12 ft. wide and 3 ft. deep, drawing 18 ins. of water. It was provided with a pile-driver, 28 ft. high, placed at one end, a propelling wheel, working in an open well, at the center, and a steam engine at the other end. The steam engine was used for propelling the scow and for operating the hammer used in driving the pipes. The rod driver was built at Buffalo and cost $2 600.

Mr. Low. EMILE LOW, M. AM. SOC. C. E. (by letter).—Mr. Pitts is correct in stating that the capping stones of the stone breakwater appear to be quite regularly rectangular in form, and of about 6 to 7 tons weight. An examination of Fig. 2, Plate VIII, will bear out this statement. In this view is shown a stone weighing 17 tons, and several others nearly as heavy.

Mr. Pitts seems to think that a comparatively small movement of the harbor- or lake- side angle stones might disarrange the adjoining capping stones. Fig. 18, a cross-section of the stone breakwater, shows that only on the harbor side do the capping stones rest upon angle stones. On the lake side the initial capping stone is generally a large parallelopipedal-shaped stone, placed horizontally and well bedded. The adjoining stones are normal to the slope and in close

juxtaposition. It does not seem likely that such a movement as mentioned could easily take place. Repeated soundings, at least, have shown no such movement as yet. Mr. Low.

As far as the harbor side is concerned, the only wave action to which it is subjected is that of the water passing over the top of the wall during violent storms, much in the same manner as water passing over a dam.

Regarding the ice formation in winter, no tendency to lift the capping stones from their beds has been observed.

Mr. Pitts, in referring to the settled portion of the breakwater, seems to think that the sliding was due to an insufficient mound of rubble at the toe of the rear slope, or that the slope of the harbor side of the breakwater was too steep. On page 125, it is stated that careful soundings were taken unremittingly, and following closely the depositing of materials at all times, in order to observe that the berms for the placing of the initial revetment stones were being built properly.

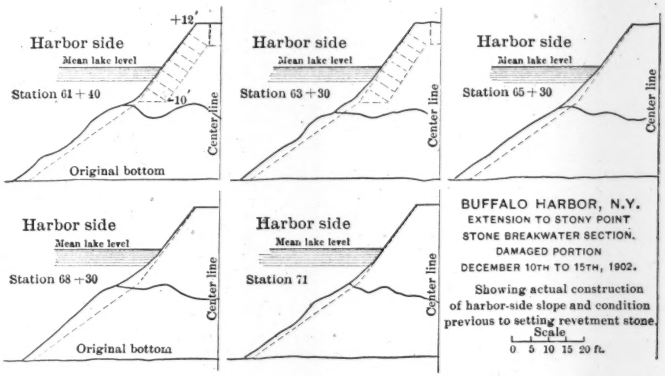


FIG. 32.

The cross-sections on Fig. 32 have been drawn from actual soundings, and show the state of construction of the harbor side at five different points of the settled breakwater. An inspection of these cross-sections makes it very evident that there was sufficient bearing for the revetment stones, the material being well filled beyond the required slopes, in many places the excess, measured horizontally, being as much as 5 ft.

It cannot be denied that gravel, on account of its form, does not pack as solidly as rip-rap or rubble, but Mr. Pitts is in error in assuming that the gravel was washed free from a portion of its matrix. The gravel was tested weekly and was found generally to contain from 25 to 30% of sand, the sieve used for sifting having a mesh of

Mr. Low. thirty-six holes per square inch. The current in the Niagara River at the place where the gravel was obtained is only a few miles per hour.

In reference to the cross-sections in Figs. 18, 20 and 22, Mr. Hughes' attention is called to page 122, where it is stated that the slopes of both the lake and harbor sides were built somewhat beyond the prescribed or theoretical lines.

It was recognized at an early date that an ample berm, especially on the harbor side, was very essential, and great care was taken to secure it.

It may be opportune to state here that the cross-sections in the paper are, to some extent, ideal, and do not, perhaps, give the absolute outline of the structure as actually built.

From Mr. Hughes' criticism it might be inferred that the gravel hearting was deposited first and that the facing of rubble was then placed upon the slope. A careful reading of page 122 will dispel this idea. These facings were the first work to be done, and consisted practically of two parallel ridges of stone, usually from 6 to 8 ft. high. On reaching this height, gravel was dumped into the space enclosed by the ridges, the depth of gravel being from 2 to 3 ft. Then more stone was dumped on the sides, raising the ridges from 2 to 4 ft. higher, when more gravel was added. This method was followed until the gravel reached a depth of from 18 to 20 ft., or 12 to 10 ft. below mean lake level, when it was covered with rubble. Although no actual cross-section could be obtained showing the exact outline of the inner slopes, in all probability it would not vary much from the sketch, Fig. 33, showing layers of gravel overlapped by rubble. The term, "rubble facing," is a misnomer; the proper appellation should have been "retaining walls."

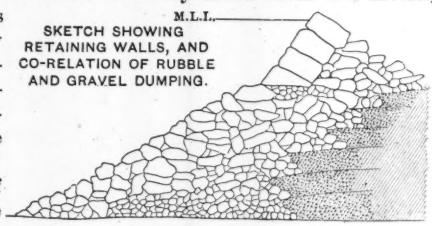


FIG. 33.

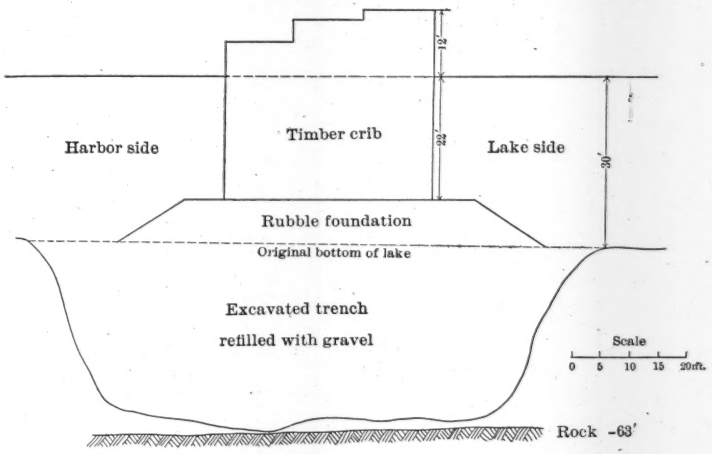
The lower parts of these retaining walls were built of small rubble, their largest dimension being 12 ins., and their weights ranging from 50 to 150 lbs. (see page 122). The remainder of the walls was built of mixed large and small rubble (see page 123), composed of pieces ranging from 200 lbs. to several tons in weight. On the lake side, these walls were covered with stone (see page 126), which served as a foundation for the revetment stone. Naturally, the interstices were small, so that there could be no agitation of water to a great extent.

On page 136 will be found the statement that "a length of about 1 000 ft. sank into the clay about 10 to 14 ft." This statement expresses the first impression, before any soundings or examinations

had been made. For the sake of literal accuracy, this statement should read thus: Mr. Low.

"On December 10th, 1902, during a violent storm, a length of about 1 000 ft. of the almost completed breakwater sank, the finished superstructure settling from 10 to 14 ft., so as to bring the original top to about the level of the water surface (a little above and a little below); the harbor slope at the same time sliding or moving outward (toward the shore) from 20 to 40 ft. at the bottom."

Regarding the steepness of the slopes of the adjacent trench for the South Harbor timber breakwater, the slopes were formed by the



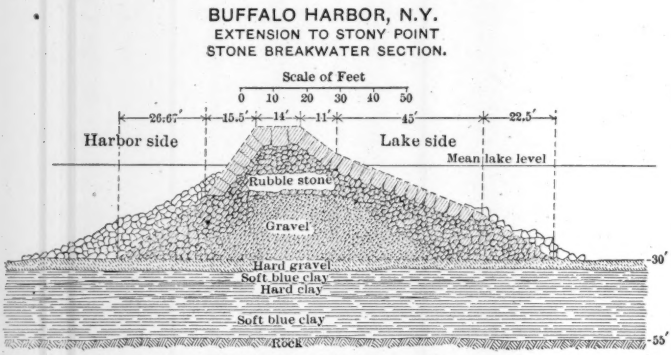
SOUTH HARBOR SECTION. STATION 72+20
FIG. 34.

method of dredging. Fig. 34 is a cross-section of this trench at Station 72 + 20, almost adjoining the sunken work. The excavated slopes are almost 2 on 1.

Some thirty test borings were taken closely along the line of the breakwater. They were about 500 ft. apart, and about the same distance shoreward from the finished structure. Between Stations 60 and 72, covered by the sunken work, the material found was soft red clay and very soft red clay. This was also the character of the material excavated for the foundation of the adjacent timber work. Many of the range piles on this portion had been forced into position merely by the weight of the hammer of the pile driver.

In accordance with the proposed plan of repairing the damage, as given on page 137, rip-rap was placed along the harbor slope. A considerable quantity was dumped on the slopes to the north of the damaged work, thus greatly reinforcing them. During the summer of

Mr. Low. 1903, much of the dredged rock from the new entrance channel off the North breakwater was also dumped on the harbor-side slope. This latter stone was deposited without additional expense to the Govern-



CROSS-SECTION AT STATION 19.
FIG. 35.

ment. Fig. 35 is a typical section of the breakwater as it now stands, between Stations 0 and 50, the shaded area showing the prescribed contract section.

On page 172 there is a reference to the memorable Galveston hurricane. The tail end of this storm struck Buffalo during the night of September 11th and the morning of September 12th, 1900. This was certainly the most severe storm which had occurred in any September, the maximum velocity of the wind, 78 miles per hour west, which occurred during the night, being the highest recorded at Buffalo during any September in thirty years. An 80-mile gale occurred on November 21st, 1900. The maximum wind velocity during the storm of December 10th, 1902, when the breakwater sank, was about 55 miles per hour.

The elevation of the water surface fluctuated with the wind during these storm periods.

As Mr. Raymond aptly remarks, the Buffalo breakwater was a very potent factor in the establishment of the immense steel plant by the Lackawanna Steel Company at Stony Point.

An inspection of the map of Buffalo Harbor, Plate V, shows a small indentation in the shore line of Lake Erie at this point, somewhat intensified by the projecting stony point of land, from which the locality receives its name.

This indentation, augmented by the construction of the shore arm breakwater, formed a large bay, affording considerable anchorage, and into this the extensive canal or waterway of the Lackawanna Steel

Company debouches. The Pennsylvania Railroad Company, in connection with the Buffalo and Susquehanna Iron Company, is also making extensive improvements on the shores of this artificial bay, among others being a grand canal to reach the blast furnaces recently erected. Other contemplated improvements are the construction of docks upon the lines of those in New York City, at right angles to the shore line, which will be a decided innovation, as far as Buffalo Harbor is concerned. Mr. Low.

The connecting link between this artificial bay and the old breakwater is the stone breakwater and its continuation, or adjunct, the South Harbor section, without which absolutely no connection between the two parts of Buffalo Harbor would have been possible during stormy weather.

The results which have already been manifest from the construction of the Stony Point extension have more than realized and fulfilled the wildest prophecies of its most strenuous advocates, but the greatest benefits will not accrue until the union of the two great enterprises, the Buffalo breakwater and the 1 000-ton barge canal, by means of the Niagara Ship Canal, is accomplished.

The Society is indebted to Dr. George E. Fell, for valuable historical reminiscences concerning the early construction of the Buffalo breakwater. As the only living representative of that period, his data have a peculiar interest, especially when it is stated that nearly all plans and notes pertaining to the beginning of the work were destroyed by the fire in the United States Engineer Office in 1888.

His description detailing the establishment of the low-water level of the lake is interesting, and is also news to many. This determination now constitutes what is termed mean lake level at Buffalo, although, lately, it has been demonstrated that it varies slightly from the correct level.

Although at variance with present practice, Dr. Fell's method of making borings seems to have given excellent results. The borings for the Stony Point extension were made by the water-jet process.

Mr. Lyle is right in stating that the history of Buffalo Harbor is incomplete without some description of the celebrated Black Rock controversy, and it is well that he has given the essential facts relating to this matter. For the proper understanding of the subject, reference should be made to Plate V. The present harbor of Black Rock embraces that part of the Niagara River lying between the foot of Maryland Street on the south and the ship canal lock below the International Bridge on the north. It is separated from the Niagara River by Bird Island Pier, which, at the southern end, consists of a timber structure filled with stone, similar in construction to the usual breakwater work. Further north it is built of rip-rap covered with broad, heavy flagging compactly laid; other parts are built of cribs filled with

Mr. Low. stone and covered with stone coping. Near Squaw Island the construction is in the nature of a levee, the eastern shore forming its boundary. Black Rock Harbor, for the greater part of its length, is separated from the Erie Canal by a division wall.

The proposed Niagara Ship Canal, mentioned previously, is the virtual improvement of Black Rock Harbor and the adjacent Erie Canal, by removing the division wall between them, and deepening this and other portions to 23 ft. below mean lake level; also the substitution of a larger lock for the present one at the lower end of Squaw Island. This, coupled with some dredging in Niagara River, will give a deep-water navigation to Tonawanda, the proposed terminus of the 1 000-ton barge canal.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 972.

A DESIRABLE METHOD OF DREDGING CHANNELS THROUGH RIVER BARS.*

By S. P. Maximoff, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Rudolph Hering, William W. Harts, E. L. Corthell, A. W. Robinson, L. J. Le Conte, F. B. Maltby, William Gerig, F. H. Hilliard and S. P. Maximoff.

The improvement of rivers by dredging is now being done on a large scale in Russia, and even a small improvement in dredging processes would result in an appreciable saving of money.

Before entering into any details regarding the writer's proposed method, he believes it necessary to give a short description of the present conditions of river navigation in Russia and of the Government's dredging fleet.

Russian River Fleet.

The last census of the Russian river fleet, taken in 1900, registered 26 154 boats and 35 000 miles of navigable waterways in European Russia.

Compared with the fleets of other countries, the tonnage is represented by the figures in Table No. 1, published recently by Mr. Lebedeff.†

* Presented at the meeting of Dec. 2d, 1903.

† *Journal* of the Ministry of Ways of Communication, 1903, V.

TABLE No. 1.

Years.	Fleet, and Country.	Metric tons.
1900	River fleet, European Russia	11 130 562
1900	Marine fleet, Great Britain	10 751 392
1897	River fleet, Germany	3 427 201
1900	Marine fleet, Germany	1 737 798
1892	River fleet, France	2 966 230
1900	Marine fleet, France	957 756

Classified by the material of construction, river boats are distributed as shown in Table No. 2.

TABLE No. 2.

	Iron.	Wood.	Total.
Steamboats	2 872	423	3 295
Towed boats	329	22 530	22 859

The growth of the number and of the freight tonnage of the towed fleet is represented by the figures in Table No. 3.

TABLE No. 3.

Year.	Number of vessels.	Cargo capacity, in metric tons.
1884	20 095	5 932 536
1890	20 125	6 572 727
1895	20 580	8 631 646
1901	22 859	11 044 094

This growth is represented graphically by Fig. 1.

The capital invested in towed boats amounts to $36 000 000, and the annual cost of building new boats of this kind is nearly $4 000 000. The average cost of a towed barge is $1 400, and there are 1 375 barges costing more than $5 000 each.

By single river basins, the barges are distributed as shown in Table No. 4.

Fig. 1.

GROWTH OF THE RIVER-TOWED CRAFT IN EUROPEAN RUSSIA FROM 1882 TO 1900, (FINLAND NOT INCLUDED).

GROWTH OF THE RIVER STEAM FLEET IN EUROPEAN RUSSIA FROM 1815 TO 1900. (FINLAND NOT INCLUDED.)

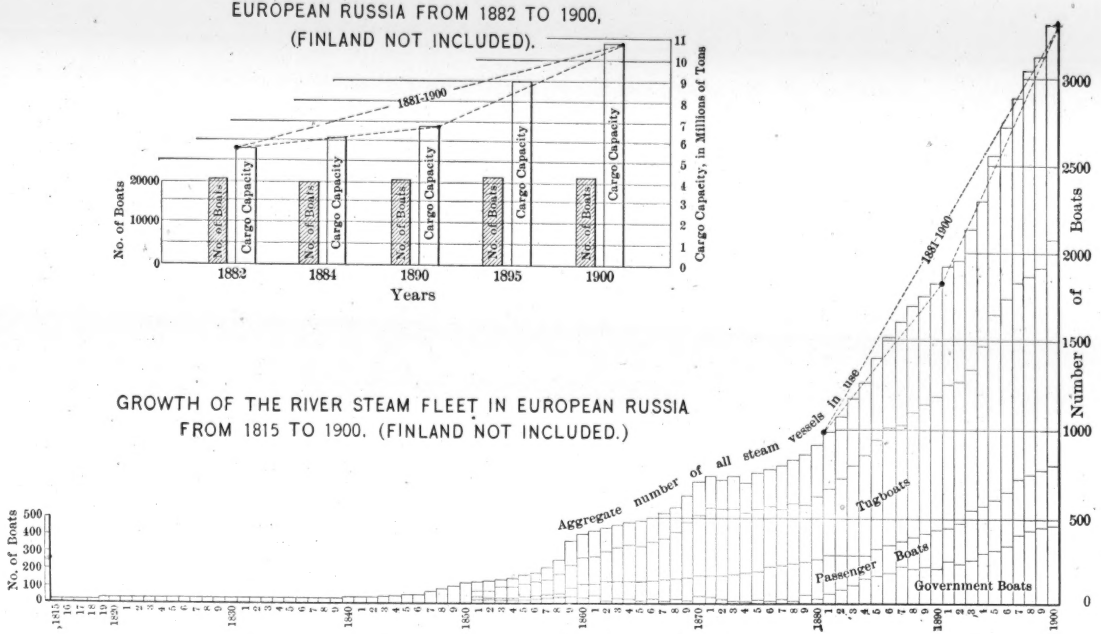


Fig. 2.

TABLE No. 4.

No.	Length of waterways, in miles.	Basins.	Number of barges.	Cargo capacity, in metric tons.
1.....	16 260	Volga.....	8 250	7 215 296
5.....	2 570	Neva, with lakes and canals.....	7 645	2 604 869
3.....	4 610	Dnieper, with South Boog.....	2 205	509 217
4.....	3 900	North Dwina.....	1 210	273 059
5.....	2 490	Don.....	488	202 116
6.....	1 290	West Dwina.....	1 041	69 814
7.....	1 360	Vistula (in Russia only).....	514	65 305
8.....	4 860	Other rivers.....	1 506	104 418
Totals..	37 340	.....	22 859	11 044 094

The cost of the steam fleet is nearly equal to $64 000 000, and the annual cost of building new steamboats amounts to $4 500 000.

The growth of the steam fleet is represented graphically by Fig. 2.

The steam vessels are distributed by single river basins as shown in Table No. 5.

TABLE No. 5.

No.	Basins.	Number of steamboats.	Total horse-power.	Average horse-power per boat.
1.....	Volga.....	1 718	440 019	256
5.....	Don.....	189	43 899	213
6.....	West Dwina.....	167	36 061	212
3.....	Dnieper, with South Boog.....	356	66 751	179
4.....	North Dwina.....	174	25 448	134
7.....	Vistula (in Russia only).....	50	5 022	91
2.....	Neva, with lakes and canals.....	525	59 499	87
8.....	Other rivers.....	116	10 084	87
	Totals.....	3 295	686 783	.....

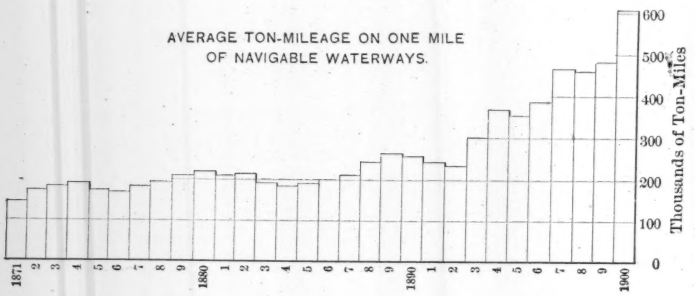


FIG. 3.

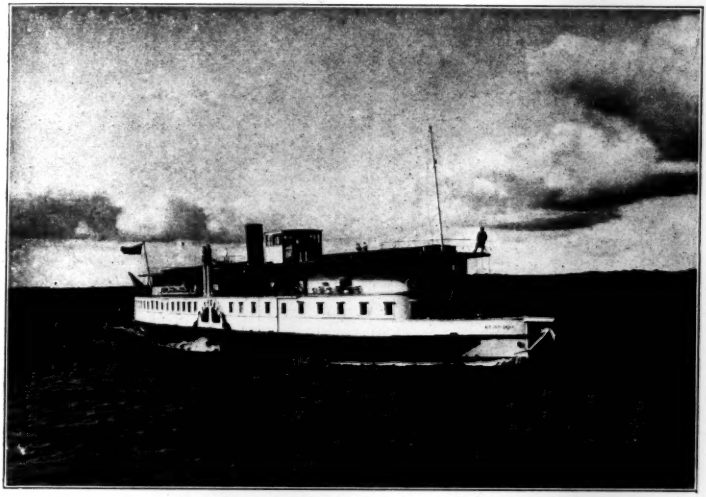


FIG. 1.—A TYPICAL RUSSIAN PASSENGER STEAMBOAT.

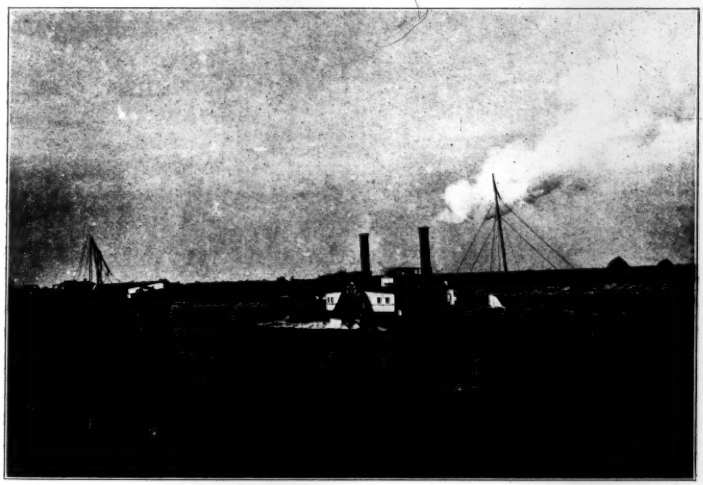


FIG. 2.—A TYPICAL RUSSIAN TUGBOAT.

The total freight capacity of these steamboats is about 1 000 000 tons.

The largest steam vessel on the Volga is the towboat, *Rededia, Prince Cassogski*, developing about 2 000 H. P., and capable of towing 16 000 tons of freight up stream.

A typical passenger steamboat is shown in Fig. 1, Plate XIV. A typical tugboat is shown in Fig. 2, Plate XIV.

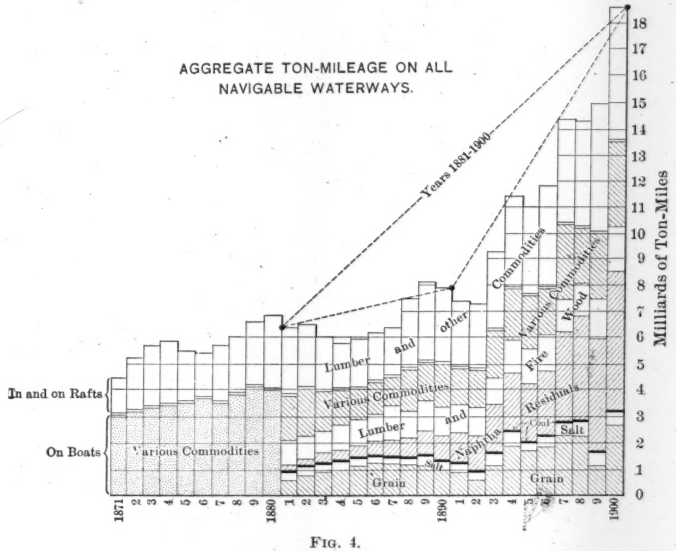
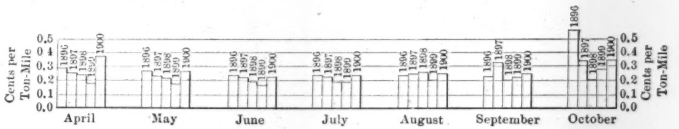


FIG. 4.

The amount of freight transported in 1900 was 600 000 tons per mile of navigable waterways (shown by Fig. 3), and the total number of ton-miles amounted to 18 600 000 000 (see Fig. 4). The rates of transportation for grain, the chief freight transported on the Volga, are shown by Fig. 5.



AVERAGE FREIGHT RATES ON THE VOLGA, TOWING UP-STREAM, FOR GRAIN TRANSPORTATION, IN CENTS PER TON-MILE.

FIG. 5.

The great progress of the water-transportation business is somewhat astonishing, when the natural conditions of Russian waterways are considered.

The Volga.

A brief statement of the physical data concerning the greatest European river, the Volga, will give a fair idea of the difficulties and obstructions to navigation.

The most important part of this river for transportation is that part between Rybinsk and Astrakhan, which can be divided into three districts.

District I.—Rybinsk to Nijni-Novgorod.—The total fall in this district is about 70 ft., distributed over a length of 296 miles.

High water rises to an average of 38 to 42 ft. above the lowest water stage, lasts nearly seventy days, and reaches the maximum at the end of April (old style). The lowest water stage arrives usually at the end of August.

The minimum discharge at low-water stage near Rybinsk is not less than 7 000 cu. ft., being generally about 15 000 cu. ft. at the end of that river district, *i. e.*, not far above the mouth of the Oka. During high water the discharge increases to 223 000 cu. ft.

The width of the summer bed of the river varies from 900 to 3 600 ft.

The depth of the pools is sometimes about 42 ft.

There are more than one hundred bars and other obstructions to navigation, and the minimum depth observed thereon during the last twenty years varies from 28 to 63 ins. But, at the same time, not more than twenty bars are dangerous for boats.

Barges in this district are from 70 to 350 ft. long, and more than 40% of the boats are longer than 280 ft. The draft of loaded boats is from 28 to 120 ins.

District II.—Nijni-Novgorod to the Mouth of the Kama.—The total length of this district is 297 miles, and the total fall about 84 ft.

The minimum discharge below the mouth of the Oka is nearly 26 000 cu. ft., and above the mouth of the Kama about 52 000 cu. ft. The maximum discharge amounts to 350 000 cu. ft.

The high-water stage lasts almost eighty-five days, and the water rises to an average height of 42 ft.

FIG. 1.—TYPICAL WOODEN BARGES ON RUSSIAN RIVERS.

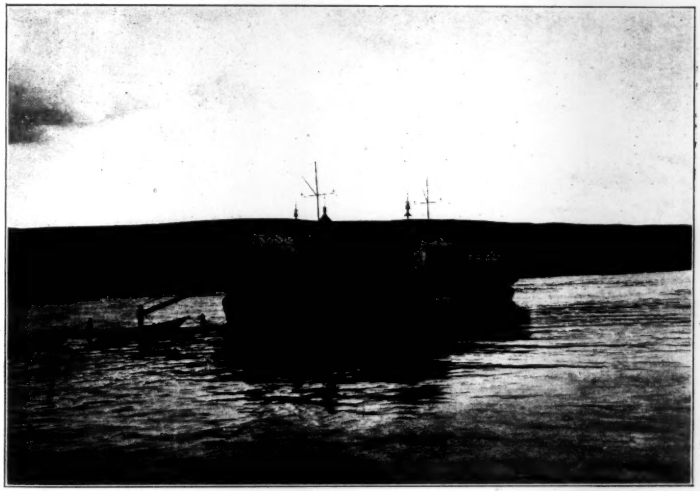


FIG. 2.—A RUSSIAN "BIELANA.

The number of bars and other obstructions amounts to seventy, and their minimum depth varies from 28 to 84 ins.

The pools at low water may be as deep as 56 ft.

The width of the summer bed of the river is from 1 000 to 3 750 ft.

Barges, here, are from 70 to 420 ft. long, and 40% of the boats are from 210 to 420 ft. long. The draft of 60% of the loaded boats is from 70 to 154 ins.

The cargo capacity of 30% of the barges exceeds 900 metric tons, and there are several barges carrying as much as 4 000 tons.

District III.—Kama to Astrakhan.—The length of this district is 1 060 miles. The fall from the mouth of the Kama to Tsaritsyn is about 165 ft.

The width of the summer bed varies from 1 750 to 5 600 ft., and sometimes reaches 9 000 ft.

The minimum flow is 113 000 cu. ft., the maximum (observed in 1884) is 1 426 000 cu. ft.

High water lasts about ninety days. It rises above the low-water stage as high as 50 ft. at the mouth of the Kama, or 12 ft. near Astrakhan.

The number of bars and other obstructions to navigation is about two hundred, and their minimum depth during the last twenty years has varied from 38 to 126 ins.

The barges, here, are from 70 to 350 ft. long, and their draft is from 28 to 140 ins. More than one-third of them draw more than 84 ins., and have a cargo capacity of nearly 3 200 tons each.

The foregoing data regarding the length and draft of towed boats refer to wooden barges—the chief freight boats on the Volga. Fig. 1, Plate XV, is a photograph of some typical wooden barges.

The maximum freight capacity of a barge, as above stated, equals 4 000 tons; but all these barges are surpassed by the "bielanas," which are boats of special construction, moved by the current, and floated down stream. They are built only for one trip, carry lumber and freight, and are floated exclusively with spring high water. Fig. 2, Plate XV, is a photograph of one of these "bielanas."

The average tonnage of these giant boats is 6 000 to 8 000 tons, and recently a mammoth one, of 13 000 tons, was built.

Fig. 6 represents the movement of freight on the Volga for the year 1900. It will be noted that a considerable part of the freight consists of rafts.

It should be remembered that the length of the period of navigation is only 6½ months. The duration of navigation and the boundaries of the river basins are shown by the map, Plate XVI.

The Government's Dredging Fleet.

The marvelous progress of the water-transportation business called urgently for the improvement of the rivers, and this has been effected by permanent structures and by dredging.

Table No. 6 gives, approximately, the appropriations for maintaining and improving the natural and artificial waterways from 1886 to 1903, not including the expenses for central and permanent local administration.

TABLE No. 6.—Appropriations for Maintaining and Improving Natural and Artificial Waterways in Russia, from 1886 to 1903.

Years.	New works.	Great repairs.	Regular annual repairs.	Building, maintaining and operating Government steamboats and dredges.	Surveying rivers and lakes, and printing the results.	Totals.
1886.....	$714 581	$820 512	$467 944	$339 354	$129 743	$2 462 184
1887.....	531 546	820 512	513 476	235 992	148 351	2 249 877
1888.....	412 820	820 512	543 302	225 928	115 384	2 117 946
1889.....	1 085 593	846 153	579 193	221 231	85 128	2 767 298
1890.....	2 012 305	1 025 640	569 334	302 460	74 833	3 984 572
1891.....	2 278 330	1 025 640	871 794		76 841	4 252 605
1892.....	1 471 793	1 025 640	620 205	302 871	56 308	3 476 817
1893.....	1 957 434	974 358	923 076		56 728	3 911 596
1894.....	1 921 798	1 102 563	1 128 204		52 718	4 205 283
1895.....	1 644 101	1 025 640	1 241 379		88 718	3 999 838
1896.....	1 341 024	1 276 922	1 261 537		87 179	3 966 662
1897.....	1 864 416	1 405 127	1 389 742		74 359	4 733 644
1898.....	2 077 116	1 138 460	1 125 979	1 167 393	79 487	5 588 435
1899.....	1 946 817	1 575 836	1 198 674	1 174 204	79 487	5 975 018
1900.....	1 136 409	1 461 537	1 538 460	2 282 746	102 564	6 521 716
1901.....	576 922	1 312 295	1 551 024	1 910 152	102 564	5 452 957
1902.....	517 435	1 538 460	1 692 306	2 564 100	102 564	6 414 865
1903.....	448 718	1 692 306	1 846 152	2 564 100	102 564	6 653 840

In the column headed "New works" are included the expenses for the improvement of the Mary Canals (from the Volga to the Baltic Sea), amounting to $7 500 000; and for the construction of the Obi-Yenisei Canal, amounting to $400 000.

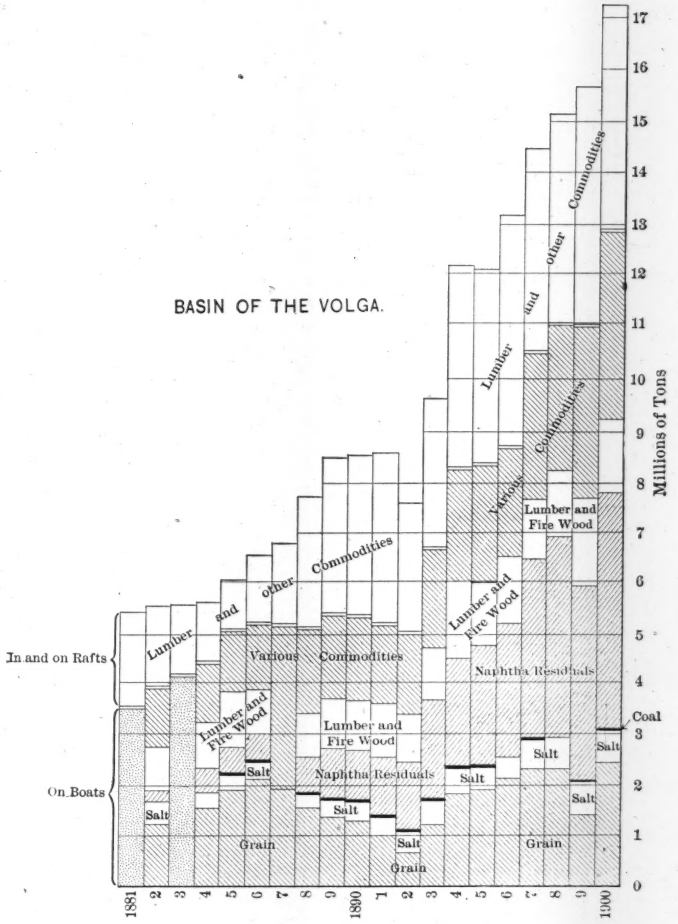

BASIN OF THE VOLGA.
Millions of Tons
In and on Rafts
On Boats
Lumber
and other Commodities
Various Commodities
Lumber and Fire Wood
Naphtha Residuals
Salt
Grain
Coal
1881
1900

FIG. 6.

These figures show clearly to what an extent the dredging operations on Russian rivers have gradually grown during the past decade.

The cost of purchasing, maintaining and operating the Government fleet of steamboats and dredges, being 11% of the total sum appropriated in 1888, rose to 36% in 1901, and 39% in 1903.

The total capacity of the Government dredges operated in 1901, as stated by contracts, was 10 510 cu. m. per hour, their number was seventy, and the capital invested in them, $3 529 421.

There were more than one hundred and fifty inspection steamboats at the disposal of the Government engineers, towing dredges, scows and snag boats, the number of snag boats being seventy.

The maintaining, operating and repairing expenses in 1901 actually amounted to $908 864.

The total material dredged was 9 261 400 cu. yds., and the total length of the cuts was 768 215 ft., or 145.5 miles.

The fuel consumption was 20 000 tons of fire wood, 6 100 tons of coal and 8 400 tons of naphtha residuals.

The average cost per cubic yard of material excavated and carried away was $9\frac{81}{100}$ cents, while for single dredges it varied from $2\frac{70}{100}$ to $48\frac{40}{100}$ cents.

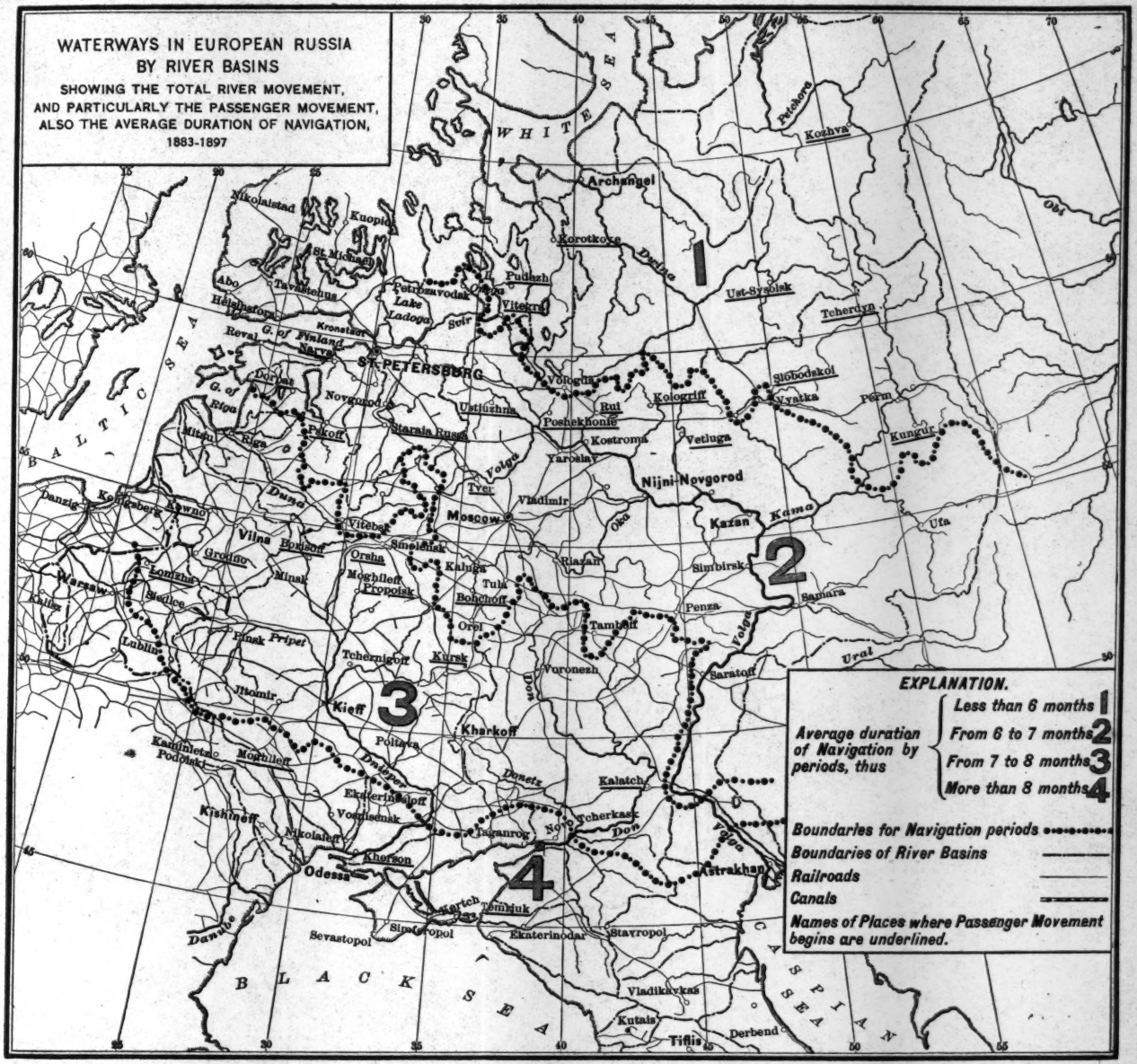
The most important dredging operations were on the Volga, as shown in Table No. 7.

TABLE No. 7.

	1899.			1900.			1901.		
	Total.	On the Volga.	Percentage.	Total.	On the Volga.	Percentage.	Total.	On the Volga.	Percentage.
Total length of the cuts, in miles.	56.9	15.4	36.9	80.1	34.9	43.6	145.5	54.2	37.3
Number of cubic yards, material excavated.	5 160 300	2 169 600	42.0	6 000 900	2 722 000	45.4	9 261 400	4 804 600	51.9
Money expended.	$516 487	$212 277	41.1	$590 700	$232 916	39.4	$908 864	$403 286	44.4
Average cost per cubic yard handled, in cents.	$10\frac{1}{100}$	$9\frac{78}{100}$	97.8	$9\frac{84}{100}$	$8\frac{56}{100}$	87.1	$9\frac{81}{100}$	$8\frac{39}{100}$	85.5

The data relating to the dredging craft on the Volga are given in Table No. 8.

PLATE XVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LII, No. 972.
MAXIMOFF ON
DREDGING RIVER CHANNELS.



The total contract-hour capacity of the sixteen Volgian dredges is 5 390 cu. m., these dredges representing an investment of $1 461 900.

It should be stated that some dredges of District II usually work on District I in the beginning of the season, together with the dredges of the last-mentioned district, because the bars in District II commence to shoal later than those in District I.

During 1901 the dredges worked in depths varying from 1½ to 12 ft. The number of hours of effective work varied from 22.6% to 80.2% of the total number of hours in the season.

On an average, the dredges spent 56% of the working time on the bars, 22% in deepening the winter refuge harbors and 22% in towing, putting in place, in boiler cleaning and other delays and unproductive work.

The dredges on the Volga are mostly ladder dredges. A typical Volgian dredge is shown on Plate XVII, and drawings of the Ladder Dredge *Volgian 9th* are shown on Plate XVIII.* They have shown very satisfactory results. The weak points are the want of power in the centrifugal pumps and the insufficient length of the floating discharge pipes. These faults are common to nearly all the dredges except the latest ones built, and will be remedied by reconstruction.

The expenses for dredging and for operating steamboats attached to the dredging craft on the Volga in 1901 amounted to $374 880. Most of the repairs of dredges and steamboats were made in the Government shops, the operating expenses of this kind amounting to $28 407. Adding this extra expenditure to the first mentioned gives $403 287, the total cost of dredging in 1901.

Assuming the amortization at the rate of 6%, the total capital invested in dredging appliances being, as above stated, $1 461 900, the annual cost of amortization will be $85 000.

The deepening of winter refuge harbors in 1901 cost $118 000, as given by Mr. Vodarski.†

Thus, for the purpose of improving the water transportation, the sum of approximately $403 287 + 85 000 — 118 000 = $370 287 was spent.

---

* This occasion is taken to acknowledge the courtesy of Mr. Ivanitski, Chief of the Waterways Department of the Ministry of Ways of Communication, who has enabled the writer to show an official drawing of the dredge *Volgian 9th*.

† *Proceedings* of the Annual Meetings of Persons Interested in Russian Waterways Development, 1902.

TABLE No. 8 (*Continued.*)

DISTRICT II.—NIJNI-NOVGOROD TO MOUTH OF THE KAMA.

Desirable depth on bars, 70 ins.

7....	1893	*Volgian 1*	Ladder.	Scows and discharge pipe.	250	3.75	215	2.15	125	220	345	89 743	Hour capacity, 4 100 cu. m. Cost $942 761.
8....	1894	*Volgian 2*	"		250	3.75	215	2.15	125	220	345	89 743	
9....	1894	*Volgian 4*	"		250	3.75	215	2.15	125	220	345	89 743	
10....	1896	*Volgian 6*	Ladder and hydraulic suction.		250	3.75	215	2.15	125	220	345	71 468	
					250	4.75	215	2.50	125	250	375	92 893	
11....	1892	*Volgian 12*	Hydraulic suction.	Discharge pipe.	100	5.68	256	2.13	.....	.....	138	72 820	
12....	1899	*Volgian 7*	Hydraulic suction.		1 500	4.26	322.5	5.47	.....	.....	1 555	263 077	
13....	1899	*Volgian 8*	Hydraulic suction.		1 500	4.26	322.5	5.47	.....	.....	1 555	263 077	

DISTRICT III.—ON THE KAMA.

14....	1862	*Kamian 1*	Ladder.	Scows.	80	3.91	.....	.....	90	.....	90	35 897	Hour capacity, 80 cu. m. Cost $35 897.

DISTRICT IV.—NEAR SARATOF.

15....	1894	*Volgian 3*	Ladder and hydraulic suction.	Scows and discharge pipe.	250	3.75	215	2.15	125	220	345	72 972	Hour capacity, 500 cu. m. Cost $150 004.
16....	1896	*Volgian 5*			250	3.75	215	2.15	125	250	375	77 032	

DISTRICT V.—THE VOLGIAN DELTA.

17....	1879	*Volgian 16*	Ladder.	Scows.	40	3.50	.....	.....	170	.....	170	28 775	Hour capacity, 40 cu. m. Cost $28 775.

TABLE No. 8.

Data Relating to the Dredging Craft on the Volga.

District I.—Rybinsk to Nijni-Novgorod.

Number.	When built.	Name of dredge.	Mode of excavating material.	Mode of transporting material.	Contract hour capacity, cubic meters.	Depth of dredging, in meters.	Length of the discharge pipe, in meters.	Height of discharge, in meters.	Indicated Horse-Power.			Cost.	Total.
									Excavating machinery.	Centrifugal pumps.	Total.		
Desirable depth on bars, 56 ins.													
1....	1862	*Volgian 13*	Ladder.	Scows	30	3.20	.....	.....	40	.....	40	$23 077	
2....	1862	*Volgian 14*	"	"	35	3.20	.....	.....	75	.....	75	26 154	
3....	1862	*Volgian 15*	"	"	35	3.20	.....	.....	75	.....	75	26 154	
4....	1900	*Volgian 11*	Ladder and hydraulic suction.	Scows and discharge pipe.	150	4.25	320	4.26	75	200	275	76 923	Hour capacity, 750 cu. m. Cost $340 371.
5....	1900	*Volgian 9*	Ladder.	Scows and discharge pipe.	350	5.50	213	2.13	275	275	550	123 077	
6....	1900	*Volgian 10*	Hydraulic suction.	Discharge pipe.	150	3.00	200	2.13	.....	.....	180	64 986	

This help to navigation is needed during low-water stage only, that is, in July, August and September.

The quantity of freight transported on the Volga between Nijni-Novgorod and the mouth of the Kama exceeds 7 000 000 metric tons, but the greater portion being carried in the spring, it is admitted that only 1 500 000 metric tons of freight are benefited directly by the dredging, so that the cost of dredging per metric ton of freight benefited is $0.243.

At the same time, the freight rate for the transportation of naphtha residuals, for example, from Astrakhan to Nijni-Novgorod (1 357 miles), is nearly equal to $1.50 per ton. It is evident that calculations of this kind are only approximate. Nevertheless, they prove clearly that navigation receives substantial aid, even from such small appropriations.

Compared with the business done, the sums expended by the Government seem to be insignificant, but they are very great when compared with the freight rates, and cannot be increased further without careful consideration. On the other hand, the fact that the appropriations are small makes it necessary to use the existing dredging craft with the greatest care and economy.

This question of economy is still more important in view of the fact that shipowners eagerly call for a further increase of the dredging fleet.

The Problem of Dredging.

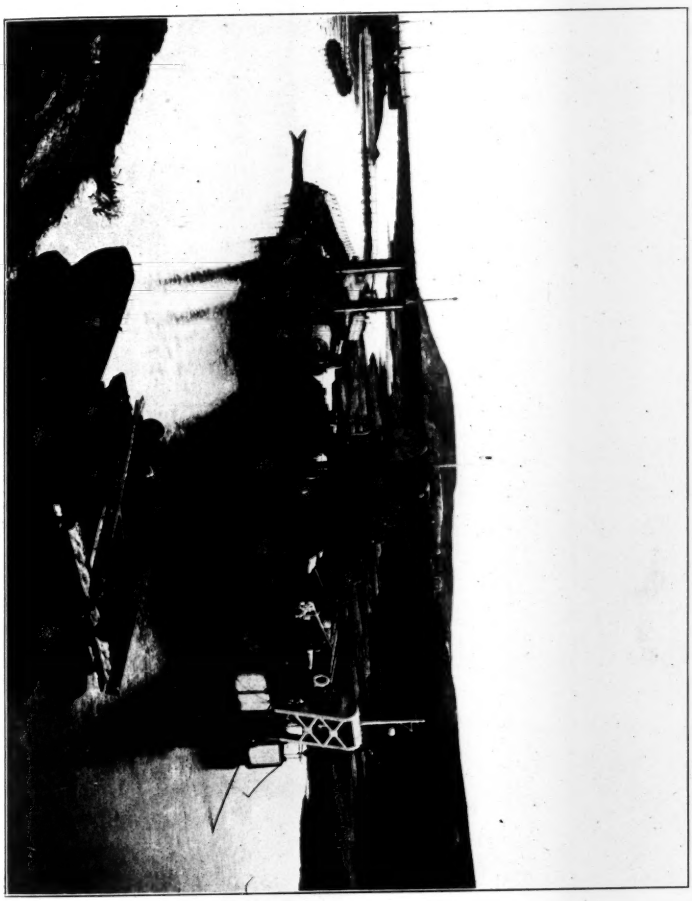
Considered from the most general point of view, the routine business of the dredging craft on rivers consists in creating an easy and deep waterway for boats, and in deepening entrances and areas of river harbors and winter refuges, all of which must be accomplished at a small cost.

To obtain this result, two things must be taken into consideration.

First, it is necessary that the dredging fleet be equipped and maneuvered with care and foresight.

Secondly, with the most modern and well-equipped plant in the world, the durability of the improvement obtained by dredging must not be lost sight of. This is a subject which has not been investigated sufficiently.

To accomplish the first requisite, the efficiency of each individual dredge in one working hour must be the maximum, under given con-



A TYPICAL RUSSIAN LADDER DREDGE.

ditions. Each dredge should be properly designed for the work to be performed, and its crew should be sufficient, but not superfluous.

The efficiency of each individual dredge during the whole working season should also be the maximum. As many hours as possible of the whole working period must be spent in excavating material and carrying it away, and idle hours, unproductive delays, towing dredges, cleaning boilers, etc., must be reduced to a minimum.

Besides this, the efficiency of the whole dredging plant must be the maximum. This means that the number of units and their capacity must be properly designed and combined; that the steamboat service be cheap enough; that the winter expenses be low; that the supervision be strongly enforced; and the accountability include all important details without being complicated.

It is not the purpose of the present short paper to explain how this service is now organized in Russia. Much has been done, in this respect, especially by Mr. Kleiber, the Second Engineer of the Dredging Department on the Volga, whose reports* have notably advanced the knowledge of this subject.

Proposed Scheme of Dredging.

It is important to find such a shape for the channel to be cut as will not only prevent any filling up, but insure its maintenance and even its scouring and deepening.

In discussing the proper shape for dredge cuts, the writer will use the data relating to the average conditions in the District of the Volga between Nijni-Novgorod and Kama. The dredge cuts on the Volga are now made as channels of constant width (250 to 260 ft.), and the depth of dropping the ladder remains constant during the whole of the operations, or is sometimes reduced at the rate at which the water level falls on the bar. To prevent a possible filling up of the dredged channel, an extra depth of 14 ins. is generally provided.

Both ways of dredging, *i. e.*, dropping the ladder to a constant depth, or raising it in proportion to the falling of the water level are left to the discretion of the dredge officers.

Although no special observations on the subject have been made, yet, according to officially published data† and special information

* See articles in the *Journal* of the Ministry of Ways of Communication; *Proceedings* of the Annual Meetings of Persons Interested in Waterways Development; and, also, other publications.

† *Supplements to Telegraphic Bulletins* of the District of Waterways in Kazan.

relating to the methods of working adopted on different dredges, the writer thinks that the dredge cuts made with a continuous raising of the ladder are durable enough during the greater part of the navigation season.

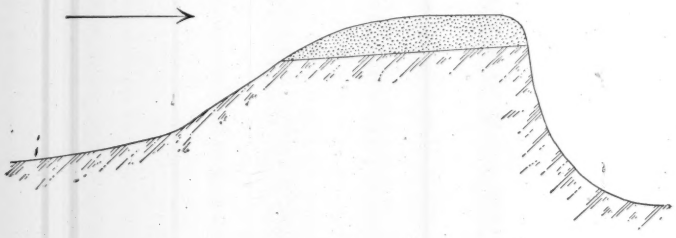


FIG. 7.

On the other hand, the writer has observed that just when the dredge begins to work, and when the first 70 to 100 ft. are left behind, the current in the dredged cut increases rapidly. Then a certain scouring out of sand is noticed, after which sand drifts commence to move at a much greater speed. All this leads him to think that it would be desirable to obtain a downward slope, in the direction of the current, on the bottom of a dredged channel.

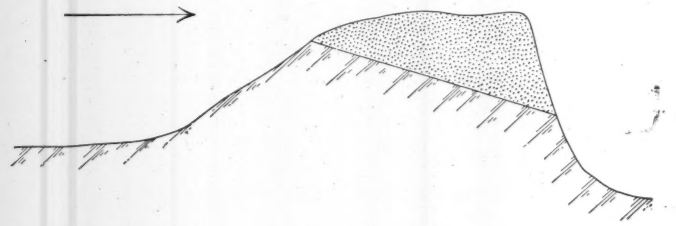
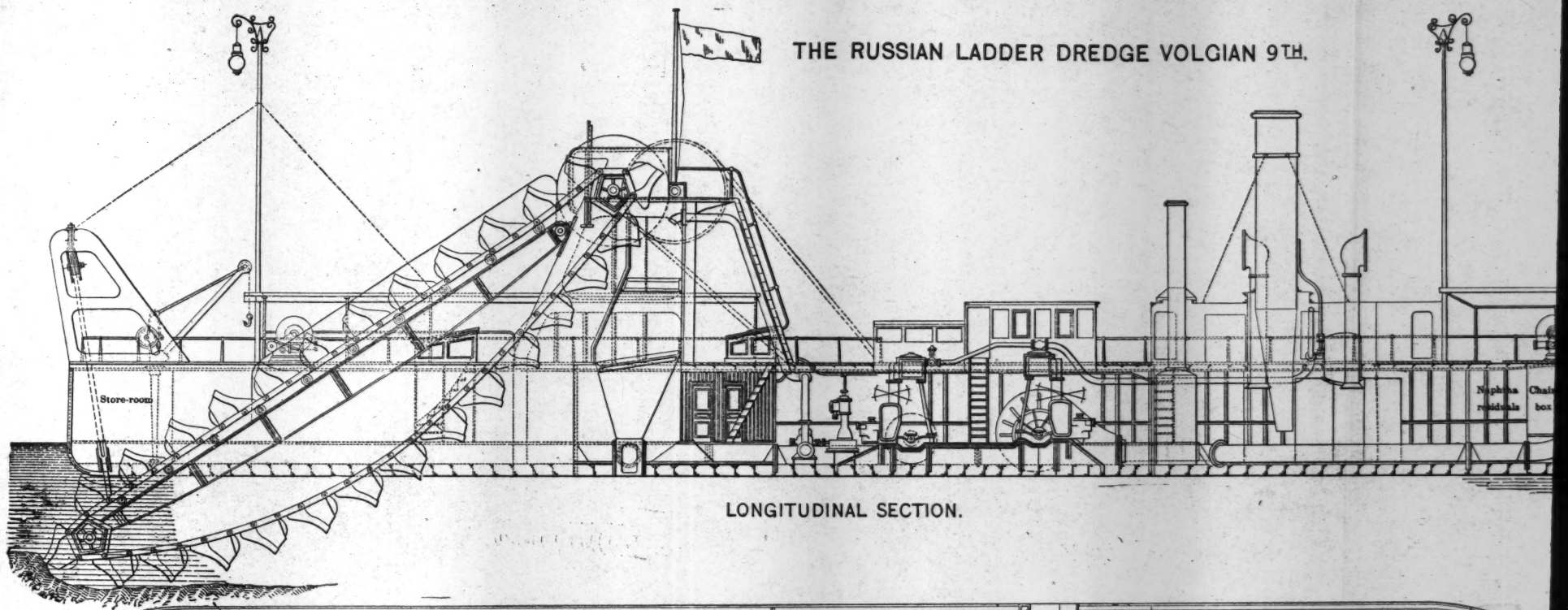


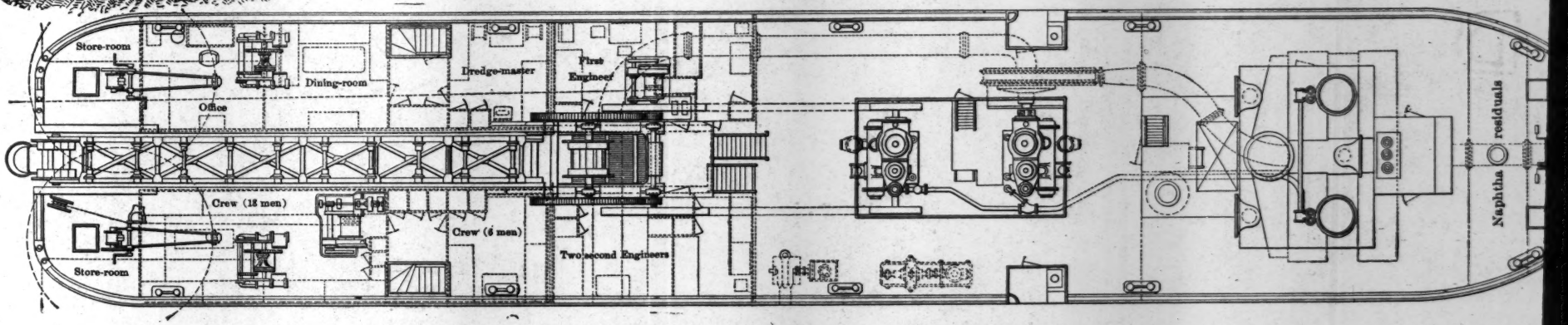
FIG. 8.

In fact, a channel in which the bottom is inclined in the opposite direction to the stream, will have, in longitudinal section, the same aspect which the bar had before the dredging operations were commenced (Fig. 7), and sand drifts, shifting on the bottom, will be impelled, as formerly, to rise upon an inclined plane; consequently, the conditions of their movement will be the same as existed previously, and the bar will tend to the former shape of equilibrium.

THE RUSSIAN LADDER DREDGE VOLGIAN 9TH.

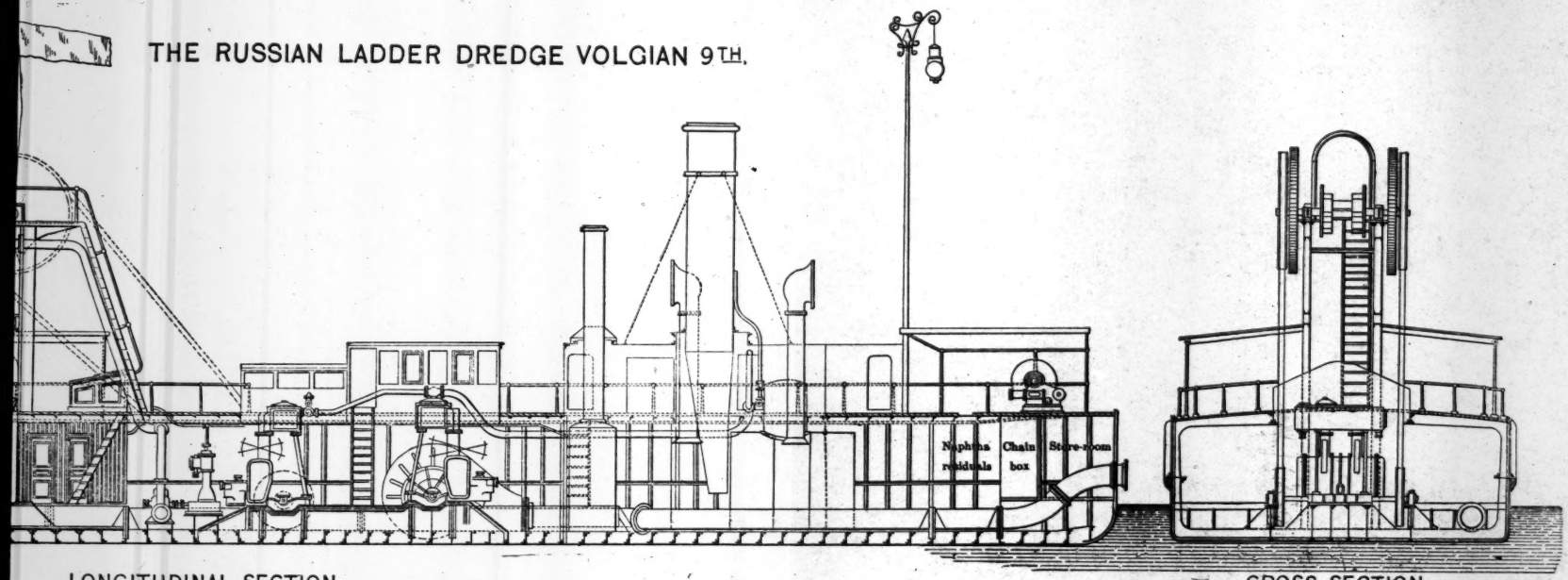


LONGITUDINAL SECTION.



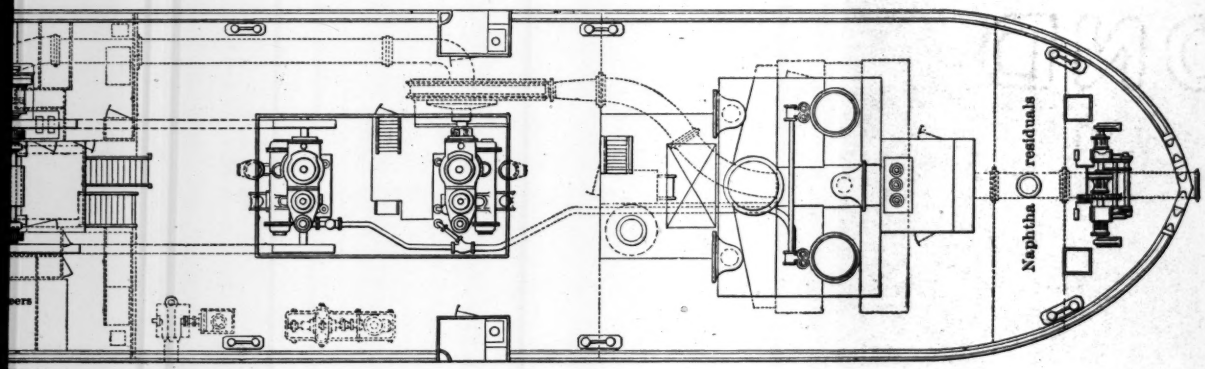
PLAN.

THE RUSSIAN LADDER DREDGE VOLGIAN 9TH.



LONGITUDINAL SECTION.

CROSS SECTION.



PLAN.

Hour capacity	350 cu. m.
Length	48.8 m.
Breadth	9.1 "
Draft	1.2 "
Depth of dredging	0 to 5.5 "
Length of discharge pipe	213 "
Height of discharge	2.13 "

Scale

1 0 1 2 3 4 5 6 7 8

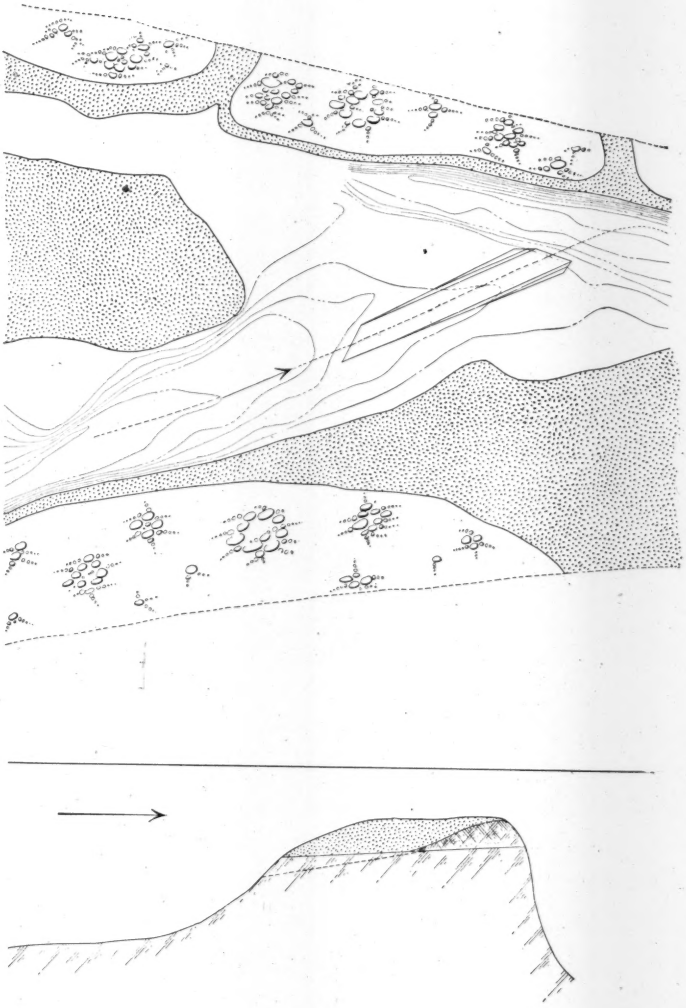


FIG. 9.

On the contrary, if the bottom of the dredged channel slopes downward in the direction of the current, the water need exert much less force in pushing the drifts down the inclined plane (Fig. 8).

It would seem, therefore, that in beginning the dredging (and moving against the current, as usual) it would be reasonable to drop the ladder as deep as the construction of the dredge permits, *i. e.*, 13 to 14 ft., and, after raising it gradually, finish the work at the depth required by navigation, *i. e.*, 70 ins. plus the height of water over its low stage. But this is not all that is necessary to ensure success.

From the examination of drawings showing dredged channels, it appears that these channels are commonly filled either from above or from below, while, in the middle, almost always, there remains a deep place which slowly disappears in the course of time.

It can be supposed that a dredged cut is filled at its down-stream end when its bottom is inclined against the current, *i. e.*, in such case the bar crest is most quickly restored (Fig. 9).

A dredged cut is probably filled at its up-stream end when it is made artificially with a downward slope in the direction of the current, or if such slope has been eroded by the current, but the width of the channel at the up-stream end remains equal to or became smaller than the down-stream end (Fig. 10).

Therefore, a dredged channel should be commenced narrow, on the supposition that it will be scoured little by little, and, according to the rate of progress of this erosion, the width of the dredged cut should be increased, going up stream, so that the funnel-shaped opening of the channel into which the water is entering shall be constantly turned against the current (Fig. 11).

Briefly, the basis of this proposed method of dredging is the supposition that, owing to the shape herein recommended, the progress of the suspended and rolling material in a dredged channel will be the easier.

This matter, and the particles of material excavated from the bottom and side slopes of the dredged channel itself, will pass along, without hindrance, to the deep water below, and the channel will be durable and possibly even deepened by scour.

Furthermore, on account of the forced erosion in the direction of the dredged cut during the progress of the work, there will be a constant reduction in the amount of cutting necessary, and the estimated total length of the cut will be reduced (Fig. 12), which is very im-

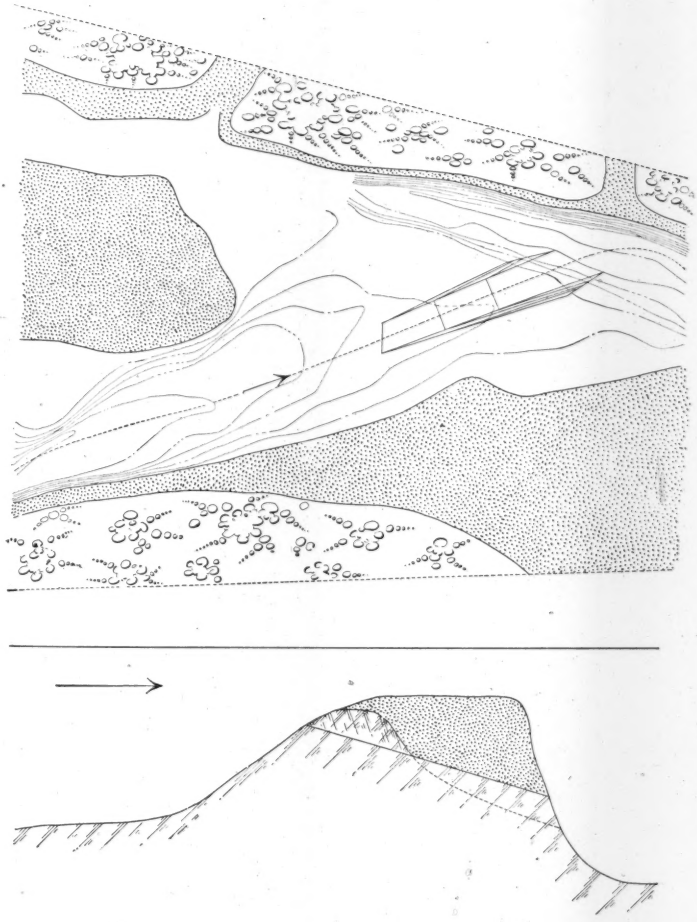


FIG. 10.

portant, in view of the fact that the dredging plant will be utilized more fully.

This reasoning may now be applied to the practical data, supposing the dredging to have been commenced after the beginning of the low-water season.

Suppose the mean length of the cut to be 1 650 ft. and the mean progress of the work to be 110 ft. a day, then the dredge will spend fifteen days in making the channel.

During that time the water will fall, roughly, from 1 to 2 ins. a day, which will make a total of 15 to 30 ins. $= F$.

Since the difference, D, between the highest and lowest positions of the ladder is $156 - 70 = 86$ ins., the fall, I, of the bottom, on the whole length of the cut, 1 650 ft., will measure

$$I = D - F = 71 \text{ to } 56 \text{ ins.}$$

This quantity is considerable in itself, but its importance becomes more evident if it is remembered that the slope of the water surface on a bar is only from 1 in. to $1\frac{3}{4}$ ins. in 1 000 ft., consequently the slope of the bottom will be $\frac{56}{1650} : \frac{1\frac{3}{4}}{1000} = 20$ times more.

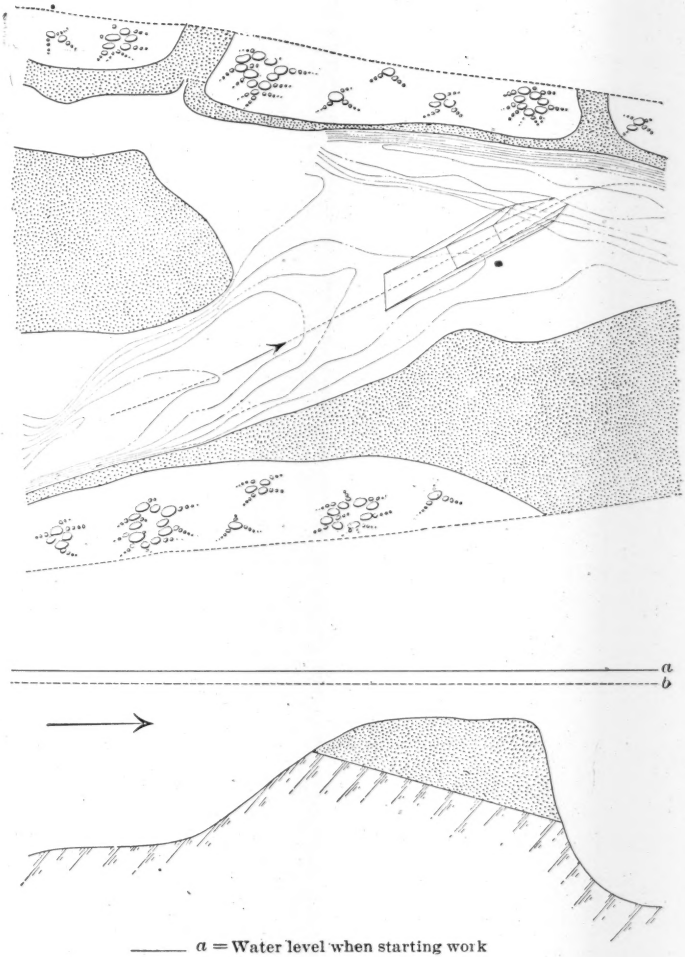
To give such a slope to the channel bottom it is necessary to raise the ladder from 71 : 15 to 56 : 15; that is, 5 to $3\frac{1}{2}$ ins. every day.

The width of the channel to be chosen beforehand can be determined approximately by supposing, for the first rough calculation, that the channel will not be scoured, and that the width of its upper end must not exceed 325 ft.

Let it be supposed that the mean velocity in the lower end of the channel must not be less than that in the upper end, and that the same quantity of water that enters the channel must pass through it. Then the areas of the channel sections at both ends must be equal, and the width of the cut must be inversely proportional to its depth, *i. e.*, the width of the down-stream end will be $\frac{6 \times 325}{13} = 150$ ft., for which width the work must be laid out.

At the end of the first day, by such a calculation, the ladder would be dropped to 12 ft. 8 ins., and the width would be extended to $\frac{13 \times 150}{12\frac{2}{3}} = 154$ or 155 ft.

At the end of the second day the ladder would be dropped to 12 ft. 4 ins., and the width would be extended to $\frac{13 \times 150}{12\frac{1}{3}} = 158$ or 160 ft.



——— a = Water level when starting work
- - - - - - - - b = " " " finishing "

FIG. 11.

It may be thought, however, that it is necessary to provide a greater width, owing to scouring at the down-stream end of the cut, but it will not be difficult to make a new cut of greater width in case erosion does not take place as expected. In all cases it is desirable to watch the results of the work during its progress, in order to determine in each case whether any modifications of the plan are necessary.

The rule to be observed is that each succeeding up-stream cross-section of the channel must be broader and shallower than each preceding down-stream cross-section.

During the progress of the work, the down-stream cross-section may be eroded, and then the above-mentioned rule must be applied to the changed conditions.

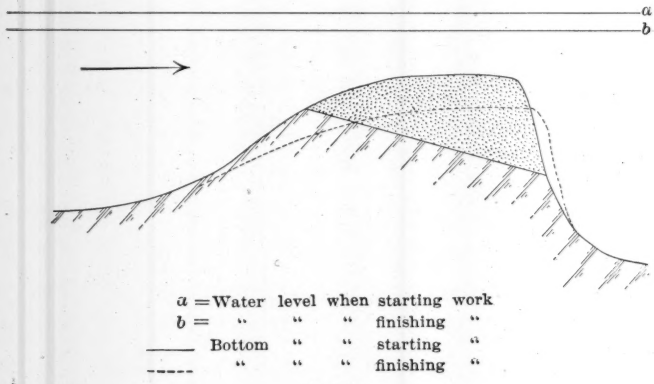


FIG. 12.

There is one feature which is essential to the success of the proposed system of dredging to which attention should be especially called: During the operations the axis of the channel must make as acute an angle as possible with the mean direction of the current through the bar. The direction of the current can be determined by the plotting of the survey showing the contour lines.

Should the dredged channel be traced so that the current erodes only one side of the cut and causes drift to form on the other, no sure method of preventing the channel from filling up can be indicated. Circumstances may occur, under which the proposed shape cannot be realized, but the writer believes that in every case, when possible, its adoption will prove justifiable.

The only objection to this method ever heard by the writer was that it would involve an extra expenditure by increasing the amount of excavation. But this would be offset by the greater stability of the dredged channel. It is believed that the increase of work would not be very great, because it is highly probable that the channel would be scoured by the current during the progress of the work. Questions of this kind can be solved only by experience.

DISCUSSION.

Mr. Hering. RUDOLPH HERING, M. AM. SOC. C. E. (by letter).—The immense territory of Russia, with its comparatively small population per square mile combined with the existence of many large rivers, seems to favor a great development of river navigation in that country. During the last fifteen years Russian engineers have been giving this subject much attention. Large sums of money are being spent in regulating streams so as to secure at all seasons a sufficient waterway for barges and steamers. The long distances to be navigated, and the many smaller streams which it was desirable to bring into use as waterways, made it necessary to confine navigation to shallow-draft vessels, in the expectation that, later, when the growth of traffic demanded it, a greater depth of channel could be secured.

The present improvements, as well as those in other countries, have been made by permanent works and by dredging. The author refers only to the latter, and suggests what he calls "a desirable method of dredging channels through river bars," by giving to the channel to be cut a different shape from the usual one. In this he does not mention a fundamental proposition for all river improvements in the alluvial territory of which he speaks, namely, to retain carefully the original regimen after the improvement has been made; in other words, the equilibrium between moving and resisting forces. The opening of a channel across a bar in this situation, increasing both sectional area and velocity, must be balanced, if the channel is to remain open permanently, by the reduction of the sectional area corresponding to the new and greater velocity, where this is least expensive and where the waterway is required.

To hold the proposed channel permanently, it would therefore be necessary to build jetties or spurs from the banks, on either or both sides, thereby narrowing it sufficiently to balance the increased velocity.

With this condition, the writer believes the author's proposition will accomplish the desired purpose. This purpose is, first, to have the bottom of the new channel slope in the direction of the current, rather than to keep it level or sloping in the reverse direction. The material of the bar, if it is shifting sand, will undoubtedly be pushed along by the water more readily on a down- than on an up- grade, and thus the scour will be more effective. The second proposition is to make the dredged channel at the down-stream end narrower than at the up-stream end. This suggestion is also sound, because, if the narrowing corresponds to the deepening, the bottom velocity or scouring force in the newly dredged channel will not at first decrease at the

lower end, as it would otherwise do, but it will rather increase and therefore help to scour until the permanent regimen is established. Mr. Hering.

Therefore, there seems to be the best of reasons why the author's general proposition is justified.

WILLIAM W. HARTS, M. AM. SOC. C. E. (by letter).—The writer has followed with much interest the data presented by the author, showing the very decided increase within recent years in vessels engaged in commerce in the interior of Russia. The comparison in tonnage, made in Table No. 1, with the merchant fleets of other countries is also somewhat of a surprise. The decided increase in appropriations during the last seventeen years, given in Table No. 6, shows that the needs of this commerce are realized and that measures are being taken to meet these new requirements. Mr. Harts.

The conditions of work are not fully explained, but the average cost of dredging per cubic yard seems low for the kind of dredge used. It appears that the dredging is done by what is ordinarily known as a "ladder" dredge, while one of the cuts shows the dredged material being pumped away by means of centrifugal pumps. In other cases referred to, the dredged material is handled in scows. It is understood that this form of dredge has been tried in this country and discarded as unsuitable to our conditions. They are still being used in Manila, as they are an inheritance, from the old Spanish Government, received by the United States on the evacuation of the Philippine Islands, and not yet replaced. Their first cost is high. The large number of moving parts makes breakages and delays in repairs serious objections, while the enormous weight necessary to be moved while working and changing location makes this form inconvenient in operation. In first cost, therefore, in maintenance, and in operation, dredges of this class do not compare favorably with dredges of the dipper, grapple, and suction types used in this country.

The best methods of securing a useful increase of depth in rivers is being given much study in America, on account of the enormous value of our internal waterways. These vary in character from tidal rivers to mountain streams, including rivers like the Hudson in its lower reaches, and the Lower Columbia; rivers in alluvial basins, like the Ohio and Mississippi; steep mountain streams, like the Kentucky and Monongahela, while streams of nearly every character may be found, and many varieties call urgently for betterment.

Dredging is the layman's cure for all lack of depth. It is homœopathic in its application, in that it is symptomatic. It affords immediate relief at the point where the trouble is felt, but the causes producing the shoal places and the result (elsewhere) of the cure are not always considered sufficiently beforehand. In many rivers, dredging

Mr. Harts. through a shoal by a deep cut of any useful size would lower the water on shoals in the upper reaches, so that this method of improvement would occasion more general injury than benefit in such streams. In other cases, where the conditions producing the shoal are not modified or corrected, the improved channel soon silts up again. It may be stated broadly, therefore, that each river must be studied in its entirety, to develop properly a successful plan of improvement, and each shoal of the river, likewise, must be examined and studied separately, as well as in its relation to the remainder of the stream.

The writer heartily agrees with the author in his statement:

"It is important to find such a shape for the channel to be cut as will not only prevent any filling up, but insure its maintenance and even its scouring and deepening."

He can scarcely agree, however, with the implied suggestion that such a shape can be found that is applicable to any but exceptional cases. The art of river engineering assists in locating dredged channels where it is probable that the maximum and most durable benefit will be secured in the shortest time at the minimum cost, but there are so many conditions to be examined, so many different problems to be solved, that it seems very doubtful whether the shape of channel suggested by the author would be of any particular advantage anywhere, except in special instances, to meet special conditions. In tidal rivers it usually appears that shoal water is found wherever the flood current takes a different channel from the ebb current. Where these two currents cross or flow in the same channel, deeper water is usually encountered. In what direction should the funnel-shaped cut proposed by the author be pointed, in dredging such a shoal?

Again, shoals are found which are produced by easily discovered causes, as at the mouth of the Willamette River, where it enters the Columbia. These two rivers are in flood at different times, the Willamette at one season of the year, from heavy rains, and the Columbia at another season, from melting snow. When the Willamette is in flood, the Columbia is backed up, and a deposit of silt in the Columbia is noticed above the Willamette's mouth. When conditions are reversed, a similar shoal is found in the Willamette River. Would a funnel-shaped cut in either river accomplish anything more permanent than the usual dredged channel? It seems that the title of the paper is, perhaps, more comprehensive than was intended.

The writer's opinion of this paper is, that the statistics and data concerning the commerce of Russian rivers are of much value, as information of this class is usually hard to get, in convenient form for use, but that the method suggested for dredging river bars is desirable only in special places on special rivers.

Mr. Corthell. E. L. CORTHELL, M. AM. SOC. C. E.—The speaker desires to state, at the outset, that he is not expert in the details of dredging, and that

this paper could have been discussed much better by, say, F. B. Maltby, M. Am. Soc. C. E., Superintendent of Dredging Operations for the Mississippi River Commission; or by Professor V. E. Timonoff, M. Am. Soc. C. E., from whom it is hoped a discussion will be received, as he has been a strong advocate of dredging in Russia; or by members of the Mississippi River Commission, who have paid great attention to the subject for many years, and have evolved the methods now in use on that river. Mr. Corthell.

The speaker agrees with Mr. Hering, and with the author, that, on general principles, a dredged slope in the direction of the current would be better than a slope against the current; but, how far that is practicable, and how far, really, in practice, it will answer the purpose of preventing deposit in the cut, is somewhat doubtful. It is a question to be solved by the results of actual practice, and these are not given by the author. Evidently, it is a theory which the author has advanced from his own observations, and, as far as it is based on experience, it might well be adopted.

It is germane to the subject to take up the more general question of dredging, not in the way of a discussion of the method proposed by the author, but as developing this very important method of improving rivers.

The subject has an important history, dating back to 1880 or 1881, not only in this country, but in Russia and other countries, and old methods are being laid aside. What was considered best twenty-five years ago, in dredging or in river improvement, has, by long experience and by dealing with great forces, been subjected to change. During the past three or four years, the experience, observation and interest of the speaker, as an engineer, have been largely upon these subjects of river and harbor improvement, which he has been obliged to study, and has been called upon to give his opinions to a foreign government.

Knowing quite well what has been going on in Russia, through a quite intimate acquaintance with Professor Timonoff, who was one of the originators of this method on Russian rivers, the speaker has arrived at some conclusions, and will give a general *résumé* of the history of dredging, because it is a very important subject, in respect to the navigation of great rivers, not only those of the United States and Russia, but those of other countries.

In the first place, in reading this paper, one is impressed by the similarity between the Volga and the Mississippi. They are of about the same size; the navigable length is about the same, about 16 000 miles; the maximum discharge of the Volga is nearly 1 500 000 cu. ft. per second, which is about the same as that of the Mississippi, although there have been extraordinary conditions in the latter when the discharge has been greater; the rise and fall of the water in places is

Mr. Corthell. approximately the same, that of the Mississippi being a little greater, perhaps; and also the conditions of the bars and the depth over them, although the minimum is less on the Volga. The general features of the two rivers are very much alike; one emptying into an inland sea, and the other into the ocean—or the Gulf of Mexico. Thus the lessons learned on the Volga can be applied to the Mississippi and to other great rivers.

Another very interesting fact relates to the number of miles of navigable water in Russia. The author has given it as about 35 000 miles. Professor Timonoff, in a paper a few years ago, stated that there were 100 000 km. (62 000 miles) of navigable waterways in Russia, but, no doubt, he included Siberia. Thus, comparing the United States and Russia, as to these features, there is more or less similarity.

In reference to the changes which have taken place in these two countries during the last twenty years, by experience, in opinion and in methods, if examined and compared carefully, it will be seen that they have kept pace with one another.

An examination of Table No. 6 shows that the appropriations for permanent works have been decreasing, particularly within the last three or four years, since very large dredges have been put in operation. This table also shows that the sums expended on dredging have been increasing, and that large amounts are still required to maintain the permanent works, or old works constructed some years ago. All this indicates that the tendency is toward a larger amount of dredging, as the author has stated.

Three or four years ago the speaker went to Argentine to advise the Government as to river regulation, and then had found it necessary to study what had been done on the Mississippi, in reference to dredging, as he had never studied it before. Taking the reports of the Mississippi River Commission, from 1881 to 1900, he had examined them carefully; taking up, first, the engineering work; secondly, the constructive work; and thirdly, the results of experience and the change of opinion. These were the headings under which the investigation had been made. Probably two or three months of study were spent in getting at the results of the experience on the Mississippi, and, in glancing over the many pages of that investigation and the notes taken, one can see the changes of opinion, due to experience, taking place, just as they have been taking place in Russia. If one should examine the reports of the Commission for the last three or four years he would see that it is coming to the principle, enunciated several years ago by French engineers, and, following them, by Professor Timonoff, which is, to improve these great rivers by "dredging and the attraction of the waters"; that is, to open channels in the direction in which the waters will flow most easily, and

in that way draw the water at low river through these artificial channels for the improvement of navigation, and with the hope and expectation that they will continue in that direction, approximately, during the high-water season, and thus, eventually, make permanent channels, without any great expenditure on permanent works. Mr. Corthell.

In the speaker's opinion, the subject is of great importance, because, from its development, lessons can be drawn from the work being done in other countries.

The following quotation is from a paper by Professor Timonoff to the VIIIth International Congress of Navigation, held at Paris in 1900, in which is laid down the law governing the case. A foot-note in that paper refers to a paper, presented to the Maritime Congress of 1889, by M. Pasqueau, Chief Engineer, Ponts et Chaussées, on the "Port of Bordeaux and its Channels":

"Quite a wrong generalization of the law of hydraulics is therefore made in arguing on these rivers (the Garonne and similar rivers), as if they were mere ditches with trapezoidal section, of which one bank can influence the opposite bank. These rivers are so very wide and shallow that the action of the banks, in fixing the navigable channel, may be considered as of no importance. The navigable channel makes its way along this expanse of water without being much influenced by the banks, obeying another law, quite as general, but which is often lost sight of. I call this law, the law of the attraction of waters, and I explain it as follows:

"Let us suppose a sheet of water, very wide, with rectilinear or other banks. If there exists, in Section *a b*, a ditch on the right bank, more water will pass in *a b*, along the right bank, than along the left bank. If a natural or artificial channel, *i k*, exists further along on the left bank, more water will pass on the left than on the right. Therefore the channel, *i k*, will determine, in the direction, *b i*, an oblique current which will carry a part of the waters from the right bank on to the left bank.

"I state this fact in saying that the channel, *i k*, will attract the waters to the left bank, and that the waters, once thrown on to this bank, will remain there, whatever be the form of the bank, as long as they will not be attracted or drawn in another direction by a natural or artificial channel established on the other side. I conclude from that, that in those rivers permanent results can be obtained by dredging directed intelligently. The effect of this dredging is, not only to take away a deposit susceptible of reforming again, but really to produce a permanent modification of the bed itself."

Professor Timonoff proceeded then as follows, in the foot note:

"Regarding this same law, we find in the discussions which followed Mr. Pasqueau's report, the following discussion of the paper by Mr. Vauthier, General Engineer:

"'What precedes brings us to Mr. Pasqueau's second question, and I agree with him as to the efficiency which, in numerous cases, a pre-existing navigable channel may have in assuring a good *régime*.

"'It is certain that by longitudinal excavations, carried out intelligently, the currents may not only be attracted into a definite region of the bed, but be maintained there; and I am prepared to acknowledge that, as regards dredging in river beds, too much stress has been

Mr. Corthel. laid on this objection, regarded as irrefutable, that the natural action which had previously modeled the bed, would succeed, sooner or later, in re-establishing things in their former state. First, it is not true of the case in which the dredge makes matter disappear which was not naturally underminable; and it is also often false, as a general rule, because the new conditions created may, co-relatively, determine up- and down-stream changes, suitable for assuring the maintenance of dispositions artificially realized.'

"In 1894 another French engineer, Mr. Gerardin, at the Congress of Navigation at the Hague, brought up the same subject and dealt with it in very nearly the same way, and he was one of the earliest who had brought forward this general method, of dredging, instead of building permanent works.

"The Mississippi Commission, in their report in 1898, used the following language in closing one of the paragraphs on this subject: 'Those principles, which seem to be most applicable to our temporary works, as well as to permanent low-water improvements, are well stated by Mr. Gerardin, Engineer of the Ponts et Chaussées, in a paper on the training of rivers at low water, read at the Sixth Inland Navigation Congress, held at the Hague in 1894.'"

From these extracts it can be seen that the subject has been discussed for some time by very prominent engineers, so that it may be concluded at the present time that it is not a matter for further discussion. It is a method which experience has taught is the best for improving the navigation of rivers like the Mississippi, the Volga, the Paraná, the Rio de la Plata and others of that nature.

In 1898, Professor Timonoff, in an important paper,* wrote:

"By what means can we improve the navigable conditions of great rivers such as the Dnieper, the Volga, the Lower Danube, the Mississippi and others? Can we not find a method more sure and less costly than the methods of improvement as they are applied to-day? The author has sought to solve this problem, and a long analysis of the works of improvement executed in France, in the United States, in Germany and Russia and other countries, has led him to formulate and to propose a method which he calls 'Regulation or Improvement by Dredging and the Attraction of the Waters'; a method taught particularly by the theoretical researches of the French scientists and the experiments in dredging by American engineers."

And then he states, further:

"And this is what it consists of, *viz.:* The waters are not to be directed by force over the bars which are to be improved; on the contrary, we are to preserve those bars from the danger of being too deeply cut, as they are often, by permanent works. And the secondary channels are not to be closed, at first, at least, and not until, by means of dredging, the proper location of the channel has been determined. We commence, after a very careful study of the local conditions of each of the bars, a strong mechanical dredging in the direction of the channel considered the most suitable and the most convenient. The width of the cut is reduced to the minimum. The depth, on the contrary, should be made as great as possible, and the mechanical dredging itself is a means of improvement, or regulation, independent of its importance, and so much the more capable of making immediately a depth necessary for the navigation. And, finally, the cut

* *Annales des Ponts et Chaussées.*

made across a bar modifies the conditions of the movements of the waters, and here the law of the attraction of the waters applies." Mr. Corthell.

The foregoing quotation is given to show how the subject has grown and has been moved on in this direction by the French and Russian engineers.

Last year, the speaker was requested, by the Minister of Public Works of Argentine, to make a special examination, during the past winter, of the results of works on the Mississippi, and the methods pursued there; and, also, to learn the opinions of the engineers and make a report on the subject. He has done so; and, without going into all the reasons or the facts which had led him to his conclusions at that time, he would quote the closing paragraph of that report. This had been made after an examination of the improvements and the dredge boats on the Mississippi, and talking with the Superintendent of Dredging, Mr. Maltby, of Memphis, where the dredging fleet was laid up in winter quarters. It was as follows:

"Generally, it may be remarked as a conclusion, with respect to the whole subject of operations on the Lower Mississippi, after many years' experience, and the expenditure of probably $40 000 000, on a river like this, the best practical plan is to dredge the channels at low water, and do only such construction work as is necessary at important points to prevent the river from making radical changes, and not to attempt to confine the channel by contracting works, as the interest on the money so expended will more than do all of the dredging work required to make and maintain the channels during the navigation season of low water. In other words, the principle, enunciated by Professor Timonoff, of 'attracting the waters' is the true principle of improving such rivers as the Mississippi and the Paraná."

The report proceeds as follows:

"The latter river (the Paraná) is a much more favorable one, in respect to its conditions, than the Mississippi, for the reason that it carries only about one-tenth of the sediment carried by the Mississippi. The channels, once made in the line of the strongest currents, will not only maintain themselves for, probably, several years, but will be enlarged by thus directing the current through them. Even on the Mississippi, where immense quantities of sediment are brought down in the annual floods of the river, the channels are found to be much better maintained than was expected of them. In fact, most of them, when properly laid out to receive the current in good line, have largely maintained themselves, so that the dredge work is really becoming less and less; so that we may say that experience here and on the great rivers of Russia has demonstrated the fact that in making and maintaining river channels, the dredge work should predominate, although there are many cases on all these rivers where guiding and controlling dikes and protection of banks may be found useful and often necessary."

On the Argentine rivers, particularly at the head of the Rio de la Plata, there are found practical illustrations of what will result from dredging. The dredging, however, was not done by a dredge, but, after very careful examination of the bottom and of the direction and

Mr. Corthell. velocity of the currents, the steamers had been sent through a certain channel, marked by luminous buoys, so that by day and by night they had passed over the same ground exactly, and thus the depth over a difficult bar had been increased from 14½ to 19 ft. within two years.

This very satisfactory result and the results in other countries led the Argentine Government to adopt the same method on probably one of the most troublesome bars to be found anywhere for ocean navigation, the Punto Indio Bar in the Lower Rio de la Plata, which is 24 miles wide, on which there were only 19 to 20 ft. of water, and where the river was 35 to 40 miles wide. The plan now being carried out, after examining all the conditions carefully, ascertaining the velocities and directions of the currents, and examining the material in the bed of the river, is to buoy, by light-boats and luminous buoys, a channel over the entire bar. The buoys and light-boats can be seen from one another. It is the expectation, and the speaker believes it to be well grounded, that within, say, three or four years after this route of navigation has been used by deep-draft steamers, there will be a very considerable increase in the depth of water.

There are many rivers in the world where it is impossible to build constructive works, and the Rio de la Plata is one of them. When the speaker went to Argentine he believed that the principles which had been applied and the methods which had been used on the Mississippi, that is, guiding works, dikes, dams, etc., might be used on the Paraná and the Rio de la Plata, but when he stood in the presence of a river 30 to 40 miles wide he made up his mind that there must be a method better than such a costly one for a government which had very small means for carrying on costly works; and it was with that before him, and with the request made to give an opinion and make plans, that he had made such a very careful study of the work on the Mississippi and in Russia, and he had become satisfied, from that study and from some practical experience in such work, that the most feasible plan for navigation, and for the exchequer of the government, was to carry out the principle of "dredging and the attraction of the waters," enunciated by Professor Timonoff.

On the Volga there is one powerful hydraulic dredge, designed by Mr. Lindon W. Bates, an American engineer, and the cost of the work done by it, compared with the cost of work done by other dredges, is given in an official report, by the Engineer of the Russian Government, for 1901 and 1902. In 1901 the cost, including all expenses—probably, however, not including depreciation and interest—was 6 cents per cubic yard, on a very large quantity of work, whereas the cost by the ordinary pump dredge on the Volga had been about 10 cents. The report states that the total quantity taken out by the Bates dredge in 1902 was 2 982 598 cu. yds., at a cost of 3.6 cents per cubic yard.

A. W. ROBINSON, M. AM. SOC. C. E.—Mr. Maximoff's paper is a very useful addition to the records of the Society on account of the data it contains relating to the navigation interests of, and the means used to improve, the rivers of Russia. The nature of these rivers and the conditions of navigation upon them are not unlike those prevailing in this country, and Mr. Corthell has drawn attention to the great similarity between the Volga and the Mississippi. Each river drains a vast extent of interior country and is subject to a spring rise of from 20 to 40 ft., or more; and each carries a large quantity of alluvial matter which forms bars obstructing the channel at low water. Each, also, is ice-bound in its northern part during the winter, and the low-water navigation depth in each is similar. The method of improving the Volga, as described by the author, is different from that used on the Mississippi. On the Volga, the author states that the dredges are of the elevator type, from which the material is pumped away a short distance. In dredges of this type, one set of machinery excavates and lifts the material and a separate set of machinery disposes of it. In view of the double duty thus performed by these dredges, the quantity of work done and the cost, as given in the paper, are most creditable, and lead one to think that, if powerful hydraulic dredges of suitable design were adopted, which could perform the double operation of excavating and disposing of the material, still better results could be obtained. Mr. Robinson.

The author states that the average cost per cubic yard of material excavated and carried away was 9.81 cents, while for single dredges it averaged from 2.70 to 48.40 cents. The cost of dredging on the St. Lawrence River, where the speaker has been doing some work lately, is less than that. In the report of the Chief Engineer of the Department of Public Works of Canada for the fiscal year ending June 30th, 1902, the cost of dredging on the ship channel of the St. Lawrence, between Montreal and Quebec, is given in detail for the various localities, as executed by the various dredges. On account of the wide diversity of material, this cost varies from 1.75 to 36.69 cents per cubic yard, the average of the whole being 4.58 cents. This is for operating expenses, including ordinary repairs, but not interest or depreciation. The dredges by which this work was done are of the elevator type and the hydraulic type, but not both combined.

With regard to the arguments adduced by the author in favor of his proposed method of cutting a sloping channel, they seem to be reasonable. There is no doubt about the scouring action of the water in a dredged cut. This is due to the more rapid current. Experience on the Mississippi and on the St. Lawrence shows that the current will flow more rapidly through the increased section of a dredged cut than over the shallower portions of the river. The speaker thinks that the most that can be said for the scouring out of the dredged

Mr. Robinson.

parts, in an alluvial river subject to wide changes in all its conditions, is that the scouring action delays the filling-up process, but does not prevent it. There are exceptions to this rule, as scouring action is sometimes found to increase the depth for a time. The rules governing sand bars are like the rules for the pronunciation of the English language—there is no rule without an exception.

Like Mr. Corthell, the speaker is an advocate of dredging the channels of alluvial rivers in such a way that the scouring action of the water will aid the artificial forces. Scouring action alone, by means of training works, has not yet been found sufficient, and, in the majority of cases where dredging has been resorted to, it has accomplished the desired result. Contrary to the general impression, a dredged channel through a sand bar is not soon obliterated, but may exist for a considerable time and survive a flood season with only minor changes.

As to the special method of dredging which the author proposes, the speaker is unable to agree with him, or with the opinion expressed by Mr. Hering, that the slope of the bottom will aid the movement of the sand. The speaker does not think that a slope of about 4 ft. in 1 000 ft. is enough to produce any perceptible effect. The movement of the sand along the bottom is not due to such an inclination, because it is too slight. It is due to the movement of the water, and is not like rolling down hill. In fact, if the section of the cut thus produced is greater at the lower end one would think it would make the velocity less, and, consequently, the scouring of the sand would be less at the lower than at the upper end. There certainly will be an increased velocity in the dredged cut, as compared with the remainder of the section of the river, as has been already noted.

The author states that he finds the power on the centrifugal pumps to be insufficient. In the outline drawing of the dredge (Plate XVIII), the engines for driving the buckets of the elevator part and those for driving the centrifugal pump are both shown as being the same, and, presumably, are engines of the same size. The speaker's experience leads him to believe that the power for driving the centrifugal pump on a dredge of this type should be two or three times that required to drive the elevator chains and buckets.

It is interesting to observe the type of river steamers illustrated in the paper, and compare the practice with that on the Mississippi. The practice on Russian rivers is to separate entirely the passengers and freight, whereas, on the Mississippi, most steamers carry both passengers and freight. The trim little passenger vessel illustrated on Plate XIV does not appear to have any self-contained gangways, for approaching a bank at any point, which are such a conspicuous feature of the Mississippi boats.

It is to be hoped that Mr. Maximoff will try his experiment and then give the Society the result.

L. J. LE CONTE, M. AM. SOC. C. E. (by letter).—The tabular statements submitted by the author show conclusively the wonderful growth of inland navigation in Russia in the past decade, more particularly on the Volga, the principal river in Russia. Mr. Le Conte.

The author gives the detailed drawings of the typical dredge now used on the Volga, which proves to be a combination machine, an endless-chain bucket dredge with hydraulic attachments instead of scows for disposing of the spoil. Machines of this type have done very good service in past years, but were discarded some time ago for those of a better type, which are now in general use. Those of the new type have greater capacity for work, in a given time, with lower operating expenses, and, at the same time, with much lower expenses for repairs, so that the unit cost of doing work is very much reduced.

The writer refers to hydraulic dredges of the latest type, fully equipped with suitable cutting apparatus adapted to encounter material of all classes.

In Table No. 8, the dredges named *Volgian 7* and *Volgian 8* are hydraulic dredges with capacities of 1 500 cu. m. per hour. These, no doubt, are of the type the writer refers to, and it is natural to suppose that each is provided with a full set of cutters to enable it to cope with material of different classes.

It should be mentioned that machines of this class are sometimes limited in their usefulness, because of the fact that it is absolutely necessary to have a suitable dumping ground upon which to place the material excavated. In all river work, however, this dumping ground feature is generally available, so that this requirement is always fulfilled.

The writer was surprised not a little to note the high cost of dredging on the Volga, and, as no mention of the character of the material was made, the inference is that it was coarse and heavy. This view was not upheld, however, by noting the flat slopes per mile, *viz.:* 2.8, 3.4 and 2.0 ins., respectively, and this precludes the possibility of much heavy material.

The average cost, 8.5 cents per cubic meter, together with the average contract-hour capacity, 337 cu. m., do not speak very well for the efficiency of the dredging fleet now in use, but leave much room for improvement. At 8.5 cents per cubic meter, 337 cu. m. would cost $28.65 per contract-hour, which seems to be excessive for a grand average. With machines of the latest type much better results are easily obtained. A 1 200-H.-P. dredge, complete, in every respect, and ready for active service, costs not more than $150 000, and it can generally be depended upon to do the work mentioned in Table No. 9, in material of different classes, at the stated unit prices.

Table No. 9 is submitted with the distinct understanding that it is only intended to give general results; therefore, it is to be used with

Mr. Le Conte. a great deal of discretion. Of course, in ordinary every-day practice, these different classes of materials are nearly always intermixed, more or less, which fact greatly facilitates dredging operations. For example, sand largely mixed or interstratified with clay or mud can easily be dredged for 3 to 4 cents per cubic yard. These conditions prevail on river bars nearly everywhere, a fact too often overlooked. On the other hand, while clean sand works very well with a moderately short pipe line, yet the difficulties multiply enormously when it has to be forced through a long pipe line. This is due to the fact that the sand separates from the column of water and runs along the bottom of the pipe with an entirely independent flow of its own. The water and sand, in their passage through the pipe line, refuse to mix. Increasing the speed of the pump, by which the mean velocity of discharge is increased, seems to have little or no effect in remedying the trouble. Both the foregoing facts affect the unit prices given in Table No. 9 quite materially, so that in each case all the existing facts and conditions must be known before any comparisons can be drawn.

TABLE No. 9.

Character of material.	Engine-hours per month.	Operating expenses and repairs per month.	Quantity of material excavated per month, in cubic yards.	Cost per cubic yard, in cents.
Very hard sandy clay and cobblestones	500	$6 000	20 000	30
Sandy clay	500	5 700	25 000	23
Tough clay	500	5 300	30 000	18
Shingle	500	5 000	35 000	14
Coarse sand	500	4 750	40 000	12
Sand	500	4 500	50 000	9
Stiff mud	500	4 250	100 000	4
Soft mud	500	4 000	200 000	2

The writer was much impressed by the author's proposed scheme for dredging channels through river bars. The general custom in America has been to make the cut uniform in width and depth, and locate it where the axis of the current seems to settle for the season. The writer thinks that there is much in the suggestion advanced by the author. Everything seems to point that way.

Mr. Maltby. F. B. MALTBY, M. AM. SOC. C. E. (by letter).—The data presented by the author, on the extent of the commerce on the rivers of European Russia, are extremely interesting, and, to the writer, very surprising.

The writer, as Superintendent of Dredging Operations on the Mississippi River below Cairo, has had some experience which may be of

interest, and presents the following, not in criticism of the paper, but as possible additional information on the subject. Mr. Maltby.

On the Mississippi River, the location of the channel to be dredged across the obstructing bar is of much more importance than the shape or size of the cut to be made. An improperly located cut will not be maintained, whatever its shape or size, while, on the other hand, if the cut is located so as to get the maximum effect of the current along its axis it will probably be maintained, if not enlarged, during the low-water season. No rule for this location can be laid down. It can only be determined by study of the conditions at each individual bar, and, very largely, is a matter of experience.

Usually, when dredging is required most urgently on the Mississippi, the river is falling very rapidly, and the problem is to open up the channel as quickly as possible.

During the last dredging season the river at Cairo fell from a stage of 11.7 ft. on December 2d to 2.8 ft. on December 20th, or almost 9 ft. in eighteen days, and the rate of fall during the interval was fairly uniform. As the ruling depth which it was desirable to maintain over the bars was reached at about 7 ft. on the Cairo gauge, and as the number of dredges available just at that time was limited, it was important that work should be done as rapidly as possible. As an instance of how rapidly this work is done, the writer cites the instance of a bar which, on the arrival of the dredge, had a possible channel of 7 ft. over it. Where the dredged cut was located, the least depth was about 5.5 ft. In 68 hours after arrival at the bar a channel not less than 200 ft. in width and 12 ft. in depth had been made. The distribution of time was as follows:

Placing plant	4½ hours.
Changing cuts	3½ "
Repairs	1 hour.
Dredging	59 hours.

During this time six cuts were made having a total length of 4 440 ft., and an average depth of cut of 8 ft. The depth of channel given above is the least depth on the bar after the dredging was completed; there was a depth of about 15 ft. along the axis of the dredged cut. This channel was maintained and improved steadily both in depth and width during the remainder of the season.

Possibly, more rapid work might be cited, but the foregoing will give a fair idea of the amount and kind of work done. Evidently, under these conditions, there is no chance for any fine regulation of the depth of the suction shoe to allow for a slope on the bottom of the cut in either direction.

The writer wishes to criticise the method proposed by the author only in its applicability to the Mississippi River; theoretically, his

Mr. Maltby. reasoning seems to be sound, and, if conditions can be found which follow the assumptions, the results should be as predicted.

Mr. Gerig. WILLIAM GERIG, M. AM. SOC. C. E. (by letter).—The writer agrees with the author's statement "that the dredge should be properly designed for the work to be performed." It should not only be an efficient dredge for dredging river channels, but should also be designed so that it can be moved quickly from one bar to another without much loss of time in towing and placing the plant. On the Mississippi, it often requires only from 24 to 48 hours to dredge one bar, and then the dredge and plant must be moved quickly to another bar. This part of the design of a dredge is often overlooked, or thought of little importance, by the designing engineer.

Dredging channels through river bars is usually necessary at certain periods, or when the river is low, and on a river like the Mississippi, where a fall from 12 ft. above to the mean low-water mark has occurred in two or three weeks, the main object is to dredge the channels through the bars as quickly as possible, so that navigation will not be interfered with. When the Mississippi continues to fall slowly during a long period, many of the bars are deepened by the current and do not need dredging; but, where the fall is rapid, the bars do not cut as fast as the river falls. This increases greatly the amount of work to be done, and the number of places to be dredged. It is then necessary that the channels to be dredged be located so that a channel of proper width and depth be obtained with a minimum amount of dredging.

A dredged channel in the Mississippi, if located properly, is usually permanent during the low-water period, provided no rapid rise or fall in the stage of the river has occurred, and that obstructions, such as sunken snags, logs or barges, which are moving along on the bottom, have not lodged in or near the head of the channel. These obstructions often cause the channel to be filled. After a flood, or even after a rapid rise and fall in the river, it happens sometimes that there is a large bar in the place where, a few months previously, the channel of the river flowed.

At the beginning of the past low-water season, the least water in the Mississippi, between Cairo, Ill., and New Orleans, La., was on the bar known as "Peters Towhead Crossing", which is about 40 miles below Memphis, Tenn., where there was a depth of only 11 ft. in the channel when the river was some 16 ft. above the mean low-water mark on the Mhoon gauge. If the river could have fallen suddenly to the mean low-water mark, there would have been a bar 5 ft. high extending entirely across the river.

The Dredge *Delta*, of the Mississippi River Commission's dredge fleet, was placed on this bar at that time and dredged a channel 250 ft. wide and 20 ft. deep, by making five cuts. The width of each cut was 32 ft., and the ridges left between the cuts were removed by the cur-

rent. This channel remained for some thirty days, when, after a rise of some 4 or 5 ft., it showed signs of filling up. When the river commenced falling, the dredge was again placed on this bar at the previous location, and made five more cuts to a depth of 20 ft. As the river receded slowly, this channel was enlarged by the current, and when the river had fallen to about 1 ft. above the mean low-water mark on the Mhoon gauge, some three months after dredging, although there had been several fluctuations in the stage of the river, there still remained a 9-ft. channel of ample width. The bars on both sides of this channel were dry at the lowest water. Mr. Gerig.

It will probably be of interest to note that this dredge has a suction head arranged so that dredging can be done either up stream, or by dropping down stream over the bar. The arrangement is very simple and works well. It has been found that when the bar to be dredged is composed of ordinary river sand, the down-stream method of dredging is the more effectual, for the following reasons:

First.—That when the cut is completed, all the material has been removed, and there remains a full-depth cut. In dredging up stream the material put in suspension by the agitator is deposited in the cut under the dredge, while in dredging down stream, the material deposited under the dredge is removed as the dredge proceeds down the stream.

Second.—That when the first cut is made down stream, and extends from deep water above to deep water below the bar, there is a decided increase in the velocity of the current in the cut, which assists the dredging materially by the erosion of the sides and bottom, and thus reduces the number of cuts to be made.

Third.—That more progress is made by down-stream dredging, since there is no danger of breaking the hauling cables or machinery, and, consequently, the operator will feed the dredge to its full capacity. This is demonstrated by the record this dredge made on the "Joe Echols Bar" during December, 1903. The material was ordinary river sand, free from drift and logs, and the average depth of cut was 3.5 ft. An average of 250 ft. of cut per hour was made in dredging down stream, and 230 ft. while dredging up stream.

Fourth.—By being able to dredge both ways, much time is saved, in changing cuts and in placing the dredge, since all that is necessary is to move the dredge over to one side of the cut previously made, reverse the suction head and begin dredging.

From experience gained on the Mississippi, the writer thinks it would be impossible to lay down a prescribed method for dredging, for the reason that there are different conditions, even on the different bars on the same river, which have to be overcome. It seems to be best, however, that at least the first and last cuts should be made down stream with the current, and that, where there is ample time to

Mr. Gerig. do the work, the upper end of the channel be widened so as to funnel the water into it.

Mr. Hilliard. F. H. HILLIARD, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has read this paper and followed its discussion with the greatest interest. He agrees with the author that the greatest pains should be taken to produce a channel of regular form, the sides and bottom of which will be as even and uniform as is practicable without undue expenditure of time and money, especially on long crossings, where the length of cut is 1 000 ft. or more. He also agrees with the author's contention that a falling grade down stream would facilitate the couring action of the current, but questions whether a uniform grade of 4 in 1 000 is not too great a degree of refinement to demand for work of such a temporary nature as river dredging. In the writer's experience, soundings taken in the cut behind the dredge have varied as much as 12 ins. within 50 ft. of each other, probably owing to variation in the character of the material. With the ladder-dredges in use on the Volga, however, a much more uniform channel bottom should be obtained than with the hydraulic-jet agitator, with which the suction dredges on the Mississippi are equipped.

The effect which the action of the current will have on a dredged channel is always problematical. The writer has found that, on the Mississippi, at crossings where the observed conditions were almost identical, this action of the current varied radically. In some instances it was only necessary to cut a trench less than 100 ft. wide and let the current complete the work, while in others, apparently almost exactly similar, it was necessary to dredge the full width of the channel. Further investigation and experience will probably determine the causes of this seemingly erratic behavior.

One of the greatest obstacles to successful dredging on the Mississippi is the frequent occurrence of stumps, logs and wrack heaps in the path of the dredge. Although these, of themselves, may not offer obstruction to navigation, they interrupt the even flow of the water through the channel and produce cross-currents and eddies which cause rapid filling of dredged cuts in their vicinity.

From observation and experience, the writer has arrived at the following conclusions in regard to the fundamental requirements for a channel that will maintain itself through one low-water season:

1.—A location that is not inimical to the existing currents.

2.—A dredged channel having smooth, even and approximately parallel sides, with no offsets or sharp bends, and a smooth, even bottom.

3.—A channelway free from obstructions which would cause agitation, or deflect the current from its straight course through the cut.

4.—The upper end of the channel slightly widened to produce a funnel effect at the entrance to the cut.

Mr. Hilliard.

There is one feature of dredging operations which has not, as yet, been touched upon in the interesting discussions on this valuable paper, *viz.*, the point of first attack on a bar, and the position of the path of the dredge in making the first cut. In the most important and troublesome bars, the shoal occurs at a point where the channel crosses the river from the bight of one bend to the bight of another of opposite curvature, the lower end of the crossing approaching to within a few hundred feet of the bank. Just below the reef there is a sharp deflection of the current down stream, often as much as 60 degrees. To this change in the direction of the current, just under the reef, is due one of the most difficult obstacles to successful dredging, and also to successful navigation by heavy tows working up stream.

When the head of a dredge or boat reaches the reef line the current below the reef striking against the upper side of the boat throws its stern down stream, and the current on the crossing striking against the lower side throws the head up stream, causing the boat to move in a direction nearly parallel to the reef line. Towboat men term this action running away from a reef. The writer has seen a strong towboat, with a single barge of lumber in tow, make six or seven failures before finally succeeding in mounting such a reef, when there was abundant water on the reef to float the barge. In dredging, this same tendency to run away from a reef is encountered, and must be overcome.

The conditions shown in Fig. 13 may be considered as typical, in regard to crossings. When the dredge has been brought into position on the axis of the cut it will lie as indicated in the sketch, its longitudinal axis making a large angle with the axis of the proposed channel. When the winding engines are started, the dredge, instead of moving in a straight line toward the piles, as the uninitiated might expect, takes a course up the reef line, gradually eating its way across it, however, since the shortening cables must decrease its distance from the piles. Even after the dredge has "raised the reef," the position of the discharge pipe, acted on by the deflected current, hold its stem down and continue the tendency of the dredge to work up stream away from the axis, so that in moving to the piles it will traverse a course approximating that indicated in Fig. 13 as "course of dredge using cables alone." To overcome this influence of the deflected current, resort is made to numerous expedients. The first one that suggests itself is to sink a side pile, run a stout Manila line to it from the dredge and pull the dredge hard in against the reef while winding in the head cables. This method, like most apparent first solutions of difficult problems, has serious drawbacks. In the first place, setting the side pile and running the line to it, require time, one of the most important factors that make or mar the value of

Mr. Hilliard. dredging expedients. Secondly, the pile and line are additional obstructions to navigation, for, in river dredging, it must always be borne in mind that steamboat traffic should not be interfered with more than is absolutely necessary. Thirdly, it is not effective, for even straining a 2-in. line to the breaking point will not hold the dredge on the axis line, and a strain greater than this would be very dangerous to the telescoping joints of the suction head.

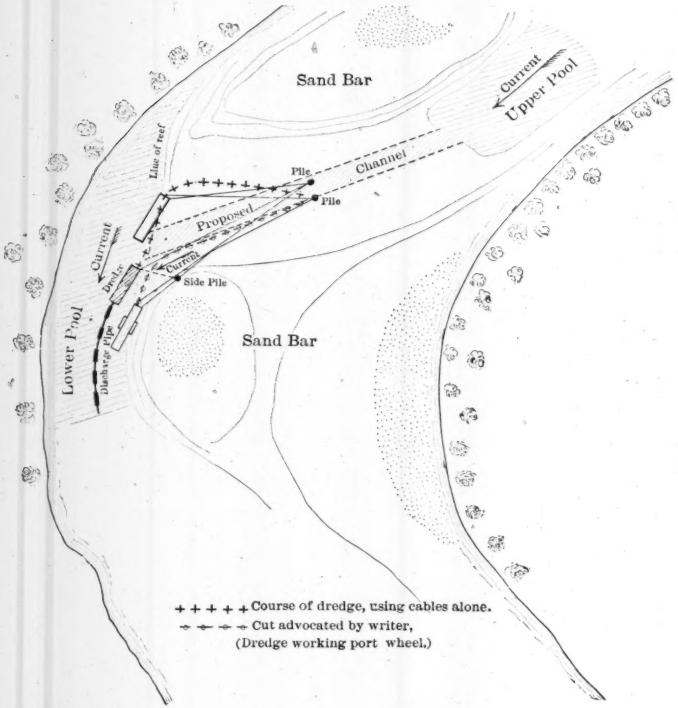


FIG. 13.

Another method, suggested and tried by the writer, was to set the head piles on the lower side of the axis. This was also partially successful in mounting the reef, but after the dredge had crossed the reef line it would swing down below the axis until the cables led directly down the current from the piles.

Where the change in direction of the current is slight, crossing the working cables, as shown, that is, running the port cable to the star-

board pile and *vice versa*, is of great assistance in holding the dredge on the desired line. Thus, referring to Fig. 13, if the starboard cable is allowed to remain slack, and the pull is made entirely on the port cable, the dredge will mount the reef close to the axis, and, after this is accomplished, the starboard cable may be tightened and the cut made along the axis to a point as close to the piles as the crossed cables will permit. When the cables are crossed, the dredge may be pulled up to within 100 ft. of the piles, whereas, with cables leading straight a farther advance of 50 ft. might be made. This disadvantage is offset, however, by the greater number of cuts that can be made from one setting of piles when the cables are crossed. Mr. Hilliard.

When the current is deflected sharply just below the reef, no one of these methods alone will give the desired results. In his experience on the Mississippi, the writer has found that by crossing the cables and commencing the first cut about 100 ft. below the lower edge of the proposed channel, the dredge can be worked through the reef on a line similar to that shown in Fig. 13 as "cut advocated by writer." In extreme cases, however, it was necessary to use additional means to force the dredge into the bar. Usually, a side pile was resorted to, with dredges not equipped with propelling machinery. The writer invariably made the first cut on the lower side of the proposed channel, as the natural tendency of the dredge is to work toward the upper side. After the first cut had been completed along the desired line, there was very little difficulty in widening this cut on the upper side, as the force of the current striking against the lower side of the dredge forces the suction into the upper bank of the previous cut. After making a sufficient number of cuts to obtain the desired width, the only work necessary to complete the channel is to cut off the triangular point left on the upper side of the channel by making a few short cuts. The rounding off of the lower corner produces a channel much more easily run by boats bound up stream, with heavy loads in tow, than one in which, throughout its length, the lower bank is parallel to the axis of the channel.

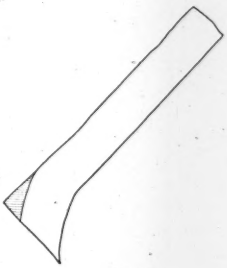


FIG. 14

Before closing, the writer desires to call especial attention to some of the advantages which the side-wheel dredges of the Mississippi River Commission of the latest type have over those not equipped with propelling machinery. Of course, the prime object in thus equipping these dredges was to avoid the necessity of providing and maintaining powerful and expensive towboats for moving the dredges to the field of operations and then from one bar to another. Apart

Mr. Hilliard. from this saving of expense, these dredges possess many other advantages over the older types. They are equipped with derricks, for handling and sinking piles, which greatly reduce the time required for placing the plant in position for work. The propelling wheels also obviate the necessity for side piles, as, by coming ahead on one or the other of these, the dredge may be worked to port or starboard as desired, the wheel, aided by the crossed cables, counteracting the flanking effect of the current.

Mr. Maximoff. S. P. MAXIMOFF, ASSOC. M. AM. SOC. C. E. (by letter).—The writer fully agrees with Mr. Hering, that jetties or spurs are desirable for the permanent improvement of a channel. But those works ought, first, to be cheap, and, second, to be buried promptly under the sediment or by unloading dredged materials around them. Now, on the Volga, below Nijni-Novgorod, the value of the lands adjoining the river, on a fair average, does not exceed $40 per acre, and the cost of a bank revetment is about $15 per linear foot. There are no levees on the Volga. Under such conditions the lands are scarcely worth protection against erosion, and it has been found more economical to let the river change its course and to obtain, by repeated dredging, the modest depth of 70 ins., which is required by navigation during the low-water period between Nijni-Novgorod and the mouth of the Kama.

The results of dredging are not permanent, and the dredging must be repeated several times, sometimes two or three times, during the season, on the same bar; yet the expenses on permanent structures on the Volga have been so heavy that dredging has really become the system of maintaining a navigable channel.

If the value of riparian lands were to increase, the conditions might be reversed.

These conditions are peculiar to the Volga and to some other great Russian rivers. The writer desires to state expressly that, considering the subject broadly, there is no difference of opinion between him and Mr. Hering.

Mr. Harts thinks that the Russian dredges are not of the best type available. The writer believes there is much room for improvement in that direction, but, when considered from a historical point of view, Russian practice may be better understood. To protect the river fleet against the spring moving of ice, steamers and barges are laid up in winter refuge harbors. Most of these harbors are not deep enough, and need dredging. The bottom is hard clay, or clay mixed with sand and mud, and is encumbered with logs, roots and similar débris. The banks are covered with bush and shrubbery. Dredging in such harbors is carried on with the same dredges as those used in the river channels, and the writer's opinion is that ladder-dredges are more suitable to meet such infinite variety of working conditions, con-

sequently they are more useful when the number of units is limited. After the dredging fleet on the Volga has grown to a considerable extent, the work can be better distributed among different machines, and hydraulic dredges are looked upon more favorably. Mr. Maximoff.

In relation to the proposed scheme of dredging, Mr. Harts points out that it is not always practicable. That was the reason for entitling the paper "a" and not "the" desirable method of dredging channels through river bars. The writer does not think his method would be of much use in the maritime part of a river, where the current is reversed daily. Most Russian rivers are non-tidal, that is, the parts which are most important for navigation are free from flood and ebb. In such rivers it is hoped that the proposed shape for a dredged cut will find a more general application.

To judge properly the case of the Columbia and Willamette confluence requires a better knowledge of those rivers than the writer has. It is possible that the funneled shape is not practicable there; but the writer willingly admits that his method is not applicable even on some bars of the Volga; he did not suggest it for all imaginable cases.

In reply to Mr. Corthell: In many respects the Volga is not so much like the Mississippi as it would perhaps appear. Fig. 15 indicates some of the differences. Looking on that figure, one is inclined to think that the Mississippi, between Columbia and Vicksburg, is raising its bed, while the Volga, near Kazan, is deepening its bed. In fact, by far the greater amount of sedimentary material in the Mississippi is transported in suspension, while dragged materials take a preponderant place on the Volga.

The mean velocity and slope of the Volga are less than those of the Mississippi. Thus the bed of the Volga is more stable. A bar has never been known to rise in a place where, a few months previously, the channel of the Volga flowed, as is the case on the Mississippi, according to Mr. Gerig. Several years would have to elapse before a change of such magnitude would obtain on the Volga. Yet it is true that the development of dredging on the Mississippi, which began nearly at the same time as on the Volga, was studied carefully by Russian engineers, and lessons learned in America were never lost sight of in Russia.

Mr. Robinson's great ability in designing dredges is quite well known in Russia. His advice, to make the power for driving the centrifugal pump two or three times as great as that required to drive the elevator chains and buckets, seems to be very sound. His opinion is that a slope of about 4 in 1 000 is not enough to produce any perceptible effect. The writer's idea is that a downward slope, however slight, is better than a slope against the current. Its proportion should depend chiefly on the construction of the dredge at work.

Mr. Le Conte gives a very interesting table of unit prices for dredg-

Mr. Maximoff. ing. It is quite natural that large dredges should work at a lower cost per cubic yard than the smaller ones, especially when the quantity to be excavated is great and the number of places where excavation is needed is limited. Their efficiency also increases and decreases with the depth of the cut. On the Middle Volga, where thirty to forty bars require dredging, the average depth of cut is from 2 to 2½ ft., only, and the quantity of excavation seldom exceeds 40 000 cu. yds.

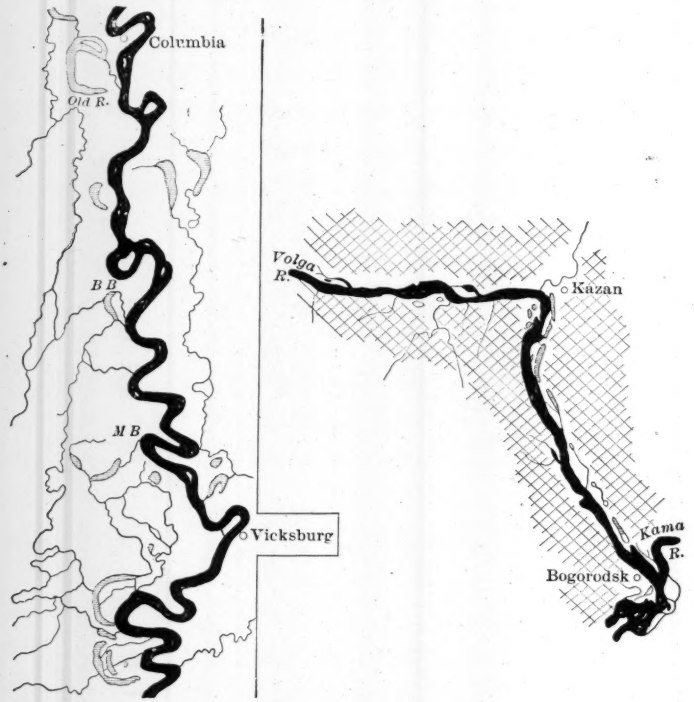


FIG. 15.

In such cases a large number of smaller units is preferable, and, in the least, will maintain the channel at less expense.

Mr. Le Conte's observation about the running of sand through the discharge pipes is important. During these last years a steady demand for increasing the length of the pipe line has been manifested, and it seems that its economical limits are nearly reached.

Mr. Maltby and Mr. Gerig furnish plenty of useful information, and the writer has no objections to make. He only wishes to point

out that the regulation of the depth of the suction shoe, to allow for a slope on the bottom of the cut, need not be too fine. The river bottom is like the fish scale, and will inevitably get such an aspect, however carefully levelled. Therefore, the suction pipe should not be raised continuously, but only by intervals. Mr. Maximoff.

The details given on the work of the Mississippi River Commission dredges are striking, and were read with greatest interest.

The grade of 4 in 1 000, mentioned by Mr. Hilliard, is an average for the bottom slope, which can be produced by dredges actually in commission on the Volga. If the ladders were lengthened, the slope could be increased, and it is possible to increase the grade, even if the cut is shorter than usual. The writer's idea is that the ladder or suction shoe should be raised continuously, so that if no scouring action occurs, the bottom of the channel will have a practically uniform grade.

Since the scouring action is uncertain, as Mr. Hilliard very properly states, the bottom will become uneven and undulated, but still it will retain the general slope given by dredging, if the location and the form of the cut are chosen properly.

Mr. Hilliard points out that the removal of obstructions in the cut is a frequent trouble. Such obstructions give much annoyance also on the Volga, not only disturbing the regular flow of water through dredged channels, but being extremely dangerous to navigation. This refers especially to anchors lost by vessels and rafts, of which quite a number is annually drawn out of the river by Government boats.

Mr. Hilliard's remarks relating to the difficulties of mounting a reef are confirmed by Volgian experience. Piles are not used, but the dredges are moored by anchors.

The question of the point of attack by hydraulic- or even bucket-dredges is a serious one in many cases; the choice is influenced greatly by considerations concerning easy movement of passing boats and harmless deposit of excavated material. Generally, it is true that making the first cut on the lower side of the proposed channel gives the best result.

The writer regrets that he is not able to mention any experiments with the proposed method, but it is now understood that they will be made.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 973.

THEORY OF THE SPHERICAL DOME WITH A HOMOGENEOUS SURFACE, AND OF THE FRAMED DOME; ALSO NOTES ON THE CONSTRUCTION OF MASONRY AND METAL DOMES.*

By E. Schmitt, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Irving P. Church, H. H. Wadsworth and E. Schmitt.

In this paper the writer presents what he believes to be original information on a subject, regarding which little or nothing has appeared in American engineering literature, as far as he has been able to ascertain. He has endeavored, also, to set out plainly the relations between the principal stresses in domes, and their magnitude; and, further, to acquaint American engineers with a method of framed-dome construction, simple in theory, and rational in execution.

Large domes of this kind exist in this country, presumably in few instances, perhaps in one instance only, at the present time, namely, in the dome of the Library of Congress, at Washington, D. C.†

*Presented at the meeting of January 6th, 1904.

†The structural design for this dome, consisting of an outer and an inner shell, was suggested and computed by the writer in 1892, under the direction of Gen. Thomas Lincoln Casey, Corps of Engineers, U. S. A., and of Bernard R. Green, M. Am. Soc. C. E., in immediate charge.

The formulas for the stresses in a homogeneous domical shell, as presented by Dr. Robison, Schwedler, and Rankine, leave their separate origin and related actions obscure, not only in their presentation, but also for the reason that they are cloaked in the abominable trigonometrical functions.*

Domes of all forms and shapes, constructed in all kinds of material, have been executed in both ancient and modern times. In the case of the older masonry domes, their execution was looked upon by contemporaries as a venturesome undertaking, an experiment inspiring awe and creating a feeling of uncertainty as to their future behavior. Later, praise and admiration were the reward for these bold and courageous designers. Modern designers should feel similarly inclined toward the old masters. The domes they built stand to-day. The Pantheon, the Santa Sophia, the dome at Florence, the dome of St. Peter's: structures, to have a chance to execute which, nowadays, would be considered by any engineer an honor gratifying his professional ambition to a high degree.

There is no doubt that the old masters knew, only in a general way, what they were doing when they built their large domes. This may be surmised from the nature of the reinforcements applied to these structures at one time or another. The nature of the "weak spot" in a dome was not known positively by them.

Some engineers have designed, or are designing, domes on principles similar to those used in the investigation of the cylindrical vault; others treat the ribs (across the diameter) of a framed dome as a three-hinged arch. Both methods are incorrect and more expensive. Correct principles were worked out some seventy years ago.

While this paper does not attempt to cover all that can be said on the subject of domes, enough will be given to enable a designer to handle a problem with intelligence. In this respect, the writer believes he is the first to suggest a method of arranging the stone coursing of masonry domes so as to do away with the defects experienced in the past, and, perhaps, give a new impetus for the execution of that monumental feature in architectural and engineering design, which has been avoided, not so much because of its cost, but

*The use of these functions in practical working formulas, the writer is bold to say, is an annoyance. If reduced to co-ordination and rectification, the conception of the problem becomes clearer, involving the whole structure, as it were, more as a being with some life than as a mere mathematical expression. A number of examples illustrating this could be presented.

because its stability has been distrusted. This, the writer believes he can overcome with the methods to be proposed.

By stating first some propositions relative to the subject, what follows will be understood more readily.

Theorem.—If, upon a circular ring, there acts a uniform radial pressure, either inwardly or outwardly, then the stress per unit length of circumference, and for a unit length of cross-section of the ring, will be equal to the product of the unit pressure by the number of unit lengths in the radius of curvature of the ring. In Fig. 1 let

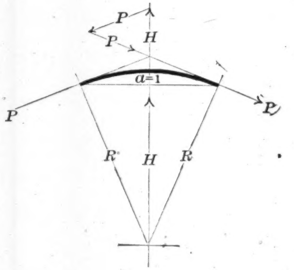


FIG. 1.

R = radius of curvature;

$a = 1$ = unit of length of circumference (for small angles its chord is assumed to have the same magnitude);

H = the unit pressure thereon;

P = the tangential peripheral stress produced thereby.

Then, $P = H R$ .... (1)

Forming the triangle of forces, as shown at the top of Fig. 1, the following proportion proves the proposition:

$$P : H :: R : a \ (= 1).$$

Surface Properties of a Sphere.—If a sphere has a radius, R, and each square unit of its surface a weight, p, then the weight, W, of any (complete) zone of the sphere, having a height, a, will be:

$$W = 2\ R\ \pi\ a\ p \quad \text{.......................} \quad (2)$$

If a zone be unity in height, then:

$$W = 2\ R\ \pi\ p \quad \text{.......................} \quad (3)$$

Considering now a spherical sector, having a base equal to unity at the equator; then a spherical square in the same sector at a parallel, having (at the base of the square) the radius, r, has a weight of:

$$W = \frac{2\ R\ \pi\ p}{2\ R\ \pi} \frac{r}{R} = p\ \frac{r}{R} \quad \text{.......................} \quad (4)$$

If, however, the length of the base of the square, at the elevation, r, be a unit in length, then the weight, W, becomes:

$$W = \frac{2\ R\ \pi\ p}{2\ r\ \pi} = p\ \frac{R}{r} \quad \text{.......................} \quad (5)$$

When the zone is not a unit in height, but such that its length, in the direction of the meridian, becomes unity, then the weight of the spherical square (see Fig. 2) will be:

$$W = p \frac{R}{r_o} \text{ sin. } i = p \frac{R\, r_o}{r_o\, R} = p \ldots\ldots (6)$$

$$\left(\text{in which sin. } i = \frac{r_o}{R}\right)$$

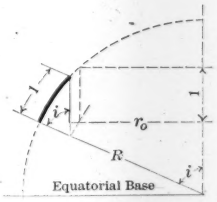


FIG. 2

Therefore, the weight of a spherical square, at any parallel having a base and a meridional height equal to a unit measure, will be

$$W = p \ldots\ldots\ldots\ldots (6)$$

The main subject will be divided, purposely, into four cases, shown and explained in Figs. 3, 4, 5 and 6. Cases I and II are the fundamental ones. By combining the effective stresses in these, the other two are solved. It will then be seen that, at any point in a complete sphere, three forces are active, and thus the place can be found where the weak part in masonry domes exists.

Case I.—Determine the stress in the crown ring. This stress, it can be inferred, is always a compressive one.

Case II.—Determine, at the base of the dome, the tangential, meridional thrust, always acting downward as a compressive stress. Determine, also, for the same place, the circumferential ring stress, *B*. This, at the bed-joint of a dome, is always tension.

Case III.—Combine Cases I and II, and the stresses at any point in a full hemispherical dome can be obtained.

Case IV.—By combining Cases I and II properly, the stresses in the crown and base rings can be found, and also those at any intermediate point between the two, in a truncated dome.

MAGNITUDE OF THE STRESSES.

Case I.—The stress per unit length of meridian, and per unit length of circumference, of the crown ring, is found by forming the triangle of forces, with p as the known quantity, as by Equation 6, and as shown in Fig. 3, comparing this with the similar triangle, composed of the dimensions of the sphere, as R, r_o and h_o, gives the proportion:

$$p : H_o : : r_o : h_o$$

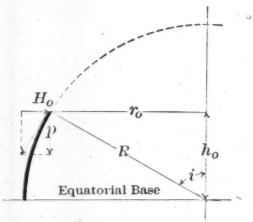
or,

$$H_o\, r_o = p\, h_o \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

By the proposition and Equation 1, the ring stress, P, in the crown ring is, therefore:

$$P = p\, h_o \ldots\ldots\ldots\ldots (8)$$

In this equation, h_o is, in each instance, the elevation of the crown ring above the (under circumstances imaginary) equatorial base of the dome.

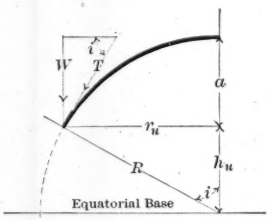


Case I—OPEN OR TRUNCATED DOME
Opening at Elevation h_o above the Equatorial Base

FIG. 3

No other force but this exists around the top of an open dome, except when a superimposed load, as that of a balustrade or a lantern, is placed on top of the crown ring, which arrangement is adopted in most of the more modern domes.

Case II.—At the base of any dome, two principal stresses exist, and must balance each other: The tangential meridional thrust, T, and the horizontal ring stress, B, in the bed-joint of the base, and which would burst the shell open if not taken care of by the tensile strength of the material of which the dome is constructed, or by some auxiliary means therein, as a metal ring or hoop. Both stresses are naturally due to the weight, W, of the dome. With this weight (see Equation 2), the triangle of forces, W, T and i, is formed (as shown in Fig. 4), and compared with the triangle of dimensions, r_u, h_u, R. Now form the proportion:



Case II—CLOSED DOME
Base at Elevation h_u above the Equator

FIG. 4.

$$W : T :: r_u : R,$$

whence

$$T = W \frac{R}{r_u} \ldots\ldots\ldots\ldots (9)^*$$

Substituting for W its value, gives:

$$T = \frac{R}{r_u} 2\, R\, \pi\, a\, p \ldots\ldots\ldots\ldots (10)$$

If B, the resisting or balancing force, as it were, is assumed to be reckoned per unit length of circumference, and for one unit length along the meridian, then, for equilibrium around the entire base of the dome, as demonstrated by Schwedler, and as will follow more directly from the "Theory of Shearing Stresses":

$$T = 2\, r_u\, \pi\, B \ldots\ldots\ldots\ldots (11)$$

* This formula is applicable, whatever may be the shape of the dome.

and, with reference to the value of T, just found:

$$\frac{R}{r_u} 2 R \pi a p = 2 r_u \pi B$$

from which,

$$B = \frac{R^2}{r^2_u} a p \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

The value, T, thus far, has been considered as the sum total of the meridional tangential thrusts all around the base of the dome; if it be also reckoned per unit length of the circumference of the base, and designated by A, then

$$A = \frac{T}{2 r_u \pi}$$

and, with reference to the value of T,

$$A = \frac{R^2}{r^2_u} a p \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

Thus it is seen that the ring stress, B, at any point around the circumference of the base of a dome, is of the same magnitude, or intensity, per unit length, as the tangential meridional thrust, A, for the same circumferential length; they are opposite in character, however, A being always a compressive, and B always a tensile stress.

Assuming the minus sign to indicate tension, and the plus sign compression, A and B can be expressed by making first the following substitutions in Equations 12 and 13:

$a = R - h$, and $r^2_u = R^2 - h^2_u = (R - h_u)(R + h_u)$, by:

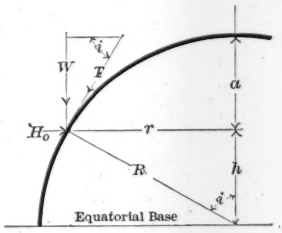
$$A = + \frac{R^2}{R + h_u} p, \qquad B = - \frac{R^2}{R + h_u} p \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Case III.—If it be assumed that the closed dome of Case II be such that when placed on top of the open dome of Case I, they, together, form a complete half sphere, or a full dome (Fig. 5), the stresses at any parallel, or latitude, having a radius, r, and an elevation, h, above the equatorial base, will be (adding Equations 8 and 14):

$$A = + \frac{R^2}{R + h} p,$$

and

$$B = - \frac{R^2}{R + h} p + p h \ldots . (15)$$



Case III.—FULL DOME, OR HALF SPHERE.
FIG. 5

Special values of A and B are:

At the crown, or the pole, where $h = R$,

$$A = +\frac{1}{2} R\,p, \text{ compression; } B = +\frac{1}{2} R\,p, \text{ also compression.} \quad (16)$$

At the base, or equator, where $h = 0$,

$$A = +R\,p, \text{ compression; } \qquad B = -R\,p, \text{ now tension...} \quad (17)$$

Equation 15, for the ring stress, B, shows it to be composed of a negative and a positive term, due to the upper and lower parts of the dome, respectively, and in reference to the parallel in question. Observing also, as just shown, that B passes from compression at the pole, to tension at the equator, it can be concluded that there exists a point, or a special value of B, where B becomes zero, or, in other words, where the two terms become equal. Therefore, putting

$$B = 0, \text{ or } 0 = -\frac{R^2}{R+h} p + p\,h$$

and solving for h, it is found that

$$h = \frac{R}{2} \pm \sqrt{R^2 + \left(\frac{R}{2}\right)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

and for $R = 1$, $h = \frac{1}{2} \pm \sqrt{\frac{5}{4}} = 0.618\ R$. $\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$

For the angle from the vertex, for which B becomes zero,

$$\text{cos. } i = \frac{h}{R} = 0.618, \text{ and } i = 51^\circ\ 49'. \ldots\ldots\ldots\ldots (20)$$

This point, at which $B = 0$, is called the principal joint of rupture. It can therefore be said: The parallel at an elevation of $0.618\ R$ above the equatorial base divides the surface of a full dome into two parts, such that all parts above it are in compression in all directions, while the lower part is in compression and in tension. The meridional stress is a compressive thrust, the ring stress a tensile one, beginning with nothing at the joint of rupture until at the base it reaches the maximum value of $B = -p\ R$.

This fact of a tensile ring stress existing in all courses below the joint of rupture, has made the (brick) masonry dome a failure, and it will continue to be a failure unless the coursings (of stone) are designed so that they can transmit tension. On the other hand, it is to be emphasized that any masonry dome extending from the pole no further than to the point of rupture is as safe and stable as any other construction designed to cover a space, as for instance a roof truss or a floor, provided there be placed around the base a ring or hoop

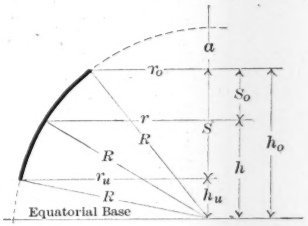
sufficiently strong to take up a tensile stress, B, or provided the base course be designed to take such a stress.

Case IV.—The Truncated Dome.—By combining properly the formulas already established, the following two general expressions for A and B, for any intermediate point, as for the parallel, r (Fig. 6), can be derived:

$$A = + \frac{R^2}{r^2} S_o p,$$

$$B = - \frac{R^2}{r^2} S_o p + p h \ldots (21)$$

Regarding the last term in the equation for B, it is to be remembered here, as in future cases, that it expresses the compressive ring stress in reference to that part of the sphere which is below the parallel in question, as by Equation 8. Therefore, when h reaches the lower limit $= h_u$, p becomes zero in that term.



Case IV.—OPEN DOME.
Opening at Elevation h_o above the Equator,
Base at Elevation h_u above the Equator,

FIG. 6

The special values of A and B are:

At crown ring,

$$A_o = 0, \qquad B_o = p\, h_o \ldots\ldots\ldots\ldots\ldots (22)$$

At base,

$$A_u = + \frac{R^2}{r_u^2} S p. \quad B = - \frac{R^2}{r_u^2} S p \ldots\ldots\ldots (23)$$

In domes, open at the top, special joints of rupture exist, and occur at points below the principal joint of rupture, provided the base of such truncated domes be also below that joint. This will appear in Figs. 16 and 19, wherein the principal stresses have been constructed graphically for several cases of domes.

Case V.—For this case, where it is assumed that an additional load, q, is imposed upon the crown ring per linear unit of its circumference, the equations for the principal stresses will also be given. Considering that a crown-ring load, Q, as it is transmitted from the top downward becomes distributed over larger and larger circles, the portion of the weight, q, at a parallel having a radius, r, will only be (Fig. 7):

$$q \frac{r_o}{r}, \text{ wherein } q = \frac{Q}{2\, r_o\, \pi} \ldots\ldots\ldots\ldots\ldots (24)$$

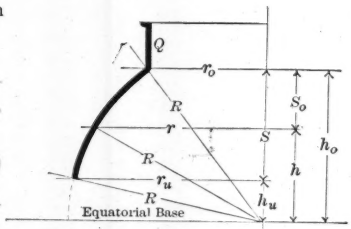
Referring to the general term for the meridional thrust,

$$A = W \frac{R}{r},$$

W becomes, in this instance,

$$W = q \frac{r_o}{r} + \frac{R}{r} S_o p \ldots (25)$$

and, in this general case of a truncated dome with a lantern load, the stresses, at any parallel, as r, will be:



Case V.—OPEN TRUNCATED DOME WITH A LANTERN OF WEIGHT Q

FIG. 7

$$A = + q \frac{R r_o}{r^2} + \frac{R^2}{r^2} S_o p \ldots\ldots\ldots\ldots\ldots\ldots (26)$$

$$B = \pm q \frac{R r_o}{r^2} - \frac{R^2}{r^2} S_o p + p_u h \ldots\ldots\ldots\ldots (27)$$

Special values:

At the crown, for $r = r_o$, $S_o = 0$, $h = h_o$, and the upper sign for B, whence,

$$A = + q \frac{R}{r_o},$$

$$B_o = + (p + q)\ h_o \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)^*$$

$$B = + q \frac{R}{r_o} + p h_o \ldots\ldots\ldots\ldots\ldots\ldots\ldots (28a)^*$$

At the bottom, at an elevation, h_u, above the equatorial base, $r = r_u$, $S_o = S$, $p_u = 0$, for $h = h_u$, therefore:

$$A = + q \frac{R r_o}{r^2_u} + \frac{R^2}{r^2_u} S p \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

$$B = - q \frac{R\ r_o}{r^2_u} - \frac{R^2}{r^2_u} S p \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

This concludes the theory of the stresses in a homogeneous spherical shell of uniform thickness, to be extended somewhat, however, in what follows. The writer is not acquainted with any treatment of this problem considering other stresses than those determined in this paper.

In passing now to the theory of the framed, or open-paneled, dome, invented and executed by W. Schwedler in 1866, it was seen that the spherical shape of the homogeneous dome surface was maintained by the ring stresses, B, counteracting, by means of the tensile strength of the material of the shell, the bursting and splitting action of the meridional thrust, A.

*While Equation 28 holds good for the cross-section of the crown ring, Equation 28a holds good for a point immediately below the same, being a circumferential shear in the bed-joint of the crown ring.

It suggested itself to Schwedler, therefore, that, if he desired to construct a dome with meridional ribs, it was proper to arrange horizontal rings, so as to tie the ribs together by hoops, as it were.

Schwedler's dome can be well likened to a masonry dome, the panels taking the places of the voussoirs; but, unlike the masonry dome, there are no open joints or interruptions in the main parts making up the construction, all stresses being transmitted by structural members connected continuously at all panel points, and designed to resist whatever kind of stresses may come upon them. Although masonry domes, being also stable structures above the joint of rupture, and in the sense that the stones then have to resist compression only, as mentioned before, the lower courses, those below the joint of rupture, are not so situated, even if the base course should be reinforced by a hoop of proper strength, the tendency of the courses in the tension zone to split open, along the meridians, would still exist.

ONE FORM OF A SCHWEDLER DOME

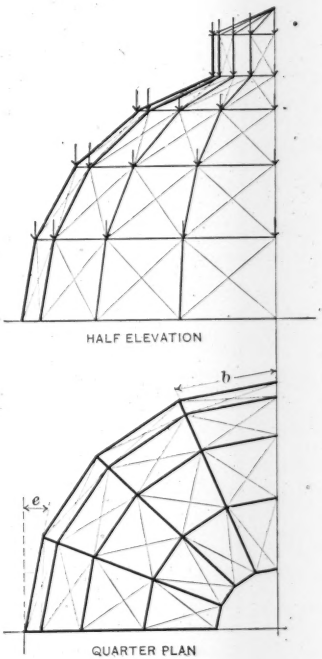


FIG. 8

In Fig. 8, one form of a Schwedler dome has been represented. From the nature of its conception, it is evident that it may be given almost any shape; the principle of its calculation once understood, there will be no difficulty in working out any special form of dome.

The spacing of the ribs and rings, generally speaking, is an arbitrary matter. When it is considered, however, that they are intended to be executed in the form of polygons, or at least to be designed in such a manner as to reduce to a minimum the compound stresses occurring in them if they are curved, and, further, that the rings have

the function of preventing a change in the form of the ribs, certain limits in the direction of the spacing of the ribs and rings cannot be exceeded. The data in Table No. 1, taken from existing examples, will give some information on this question of precedent:

TABLE No. 1.

Diameter, in feet.	Number of radial ribs.	Rise.	Number of panels.	Diameter of crown ring, in feet.	Designer.
80.....	16	1 : 8	3	Closed Dome.	Schwedler.
100.....	24	1 : 8	4	20	"
140.....	24	1 : 8	4	20	"
104.....	32	1 : 2	6	23	Library of Congress.
124.....	20†	1 : 2	7	15	Becker.

† With two jack-ribs in each main sector.

In the framed dome, as in other framed structures, the loads are assumed to be concentrated at the panel points. The determination of these loads, as indicated in Fig. 8, needs no further explanation.

As regards the stresses: The ribs are always in compression, the crown ring, also, always in compression, the foot ring always in tension. As to the intermediate ring-stresses, it is not so certain. Generally speaking, it might be said that, within an angle of 51° 49′ from the vertex, they are in compression, while below this angle they might presumably be in tension. On the other hand, it can be shown in advance how the loads must be distributed so as to increase or decrease any possible tension or compression in a ring.

In Fig. 8, conceive the topmost zone removed, then the crown ring of the lower remaining part will be in compression, and the more so the more that ring is loaded. The lower ring of the zone removed, however, if this zone be considered by itself, will be in tension, and the more so the more the topmost ring is loaded. Reasoning in a like manner for any of the rings, and the loads that might happen to come on the ring zones above them, the following laws, governing the ring and rib stresses, can be formulated:

1.—The top and bottom rings are stressed most under maximum loading.

2.—Likewise the ribs.

3.—In the intermediate rings, the greatest compression, if any, will take place when the ring itself is heavily, and all rings above it, lightly, loaded.

4.—For the greatest tension therein, if any, the reverse loading holds good.

5.—In reference to any ring, loads on zones below it do not affect the stresses in any member of the dome located above the ring in question.

6.—Any zone, from the top downward, may be removed, without disturbing the integrity of the remaining part of the dome.

7.—Inversely, the dome may be conceived to be built by placing zone upon zone, until it is completed.

Under the guidance of these rules, the following set of formulas, drawn up in reference to the diagram of a Schwedler dome shown in Fig. 9, need but little further explanation.

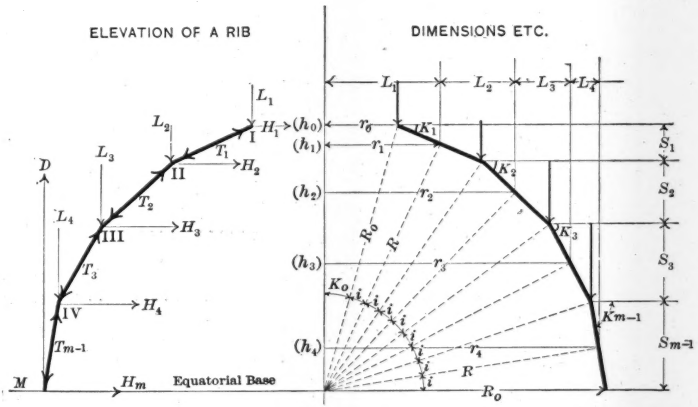


FIG. 9

Radii and elevations.			Angles and functions.	
$r_0 = R_0 \cos.\ (8\,i)$	$h_0 = R_0 \sin.\ (8\,i)$	$i = \frac{90° - K_0}{2\,(m-1)}$		$\sin.\ K_0 = \frac{r_0}{R_0}$
$r_1 = R \cos.\ (7\,i)$	$h_1 = R \sin.\ (7\,i)$	$K_1 = (K + i)$		$\sin.\ K_m = \frac{r_m}{R}$
$r_2 = R \cos.\ (5\,i)$	$h_3 = R \sin.\ (5\,i)$	$K_2 = (K + 3\,i)$		
$r_3 = R \cos.\ (3\,i)$	$h_4 = R \sin.\ (3\,i)$	$K_3 = (K + 5\,i)$		$\cos.\ K_m = \frac{h_m}{R}$
$r_m = R \cos.\ (1\,i)$	$h_{m-1} = R \sin.\ (1\,i)$	$K_{m-1} = [K + (2\,m - 3)\,i]$		

In Fig. 9 some of the dimensions (h and r) refer to the middle of the panel lengths of the main ribs, except h_o and r_o. For practical purposes, scale h and r, etc.

The ribs have each $(M-1)$ equal panels; $M = 5$ in this example.

Circumscribed radius $= R_o$.

Inscribed radius $= R_o \cos. i = R$.

Analytic Method.

Stresses in the Ribs.

Panel Point I.—In Fig. 10, the forces, L_1, the two ring forces, P_1, and the thrust in the rafter, T_1, must balance. Resolving for vertical direction:

$$L_1 - T_1 \sin. K_1 = 0$$

in which, $\sin. K_1 = \frac{r_1}{R}$

$$T_1 = + L_1 \frac{R}{r_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

Panel Point II.—Fig. 11. Similarly:

$$L_2 + T_1 \sin. K_1 - T_2 \sin. K_2 = 0$$

in which, $\sin. K_2 = \frac{r_2}{R}$

$$T_2 = + (L_1 + L_2) \frac{R}{r_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

Panel Point III.

$$T_3 = + (L_1 + L_2 + L_3) \frac{R}{r_3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

Generally:

$$T_{m-1} = +$$

$$(L_1 + L_2 + \ldots\ldots L_{m-1}) \frac{R}{r_{m-1}} \ldots (34)$$

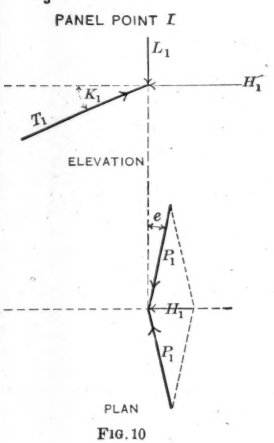


FIG. 10

Stresses in the Rings Under Dead Load.

Panel Point I.—Resolving for horizontal direction, and considering that H_1 is the resultant of the two ring forces, P_1, gives for equilibrium, Fig. 10, and with the value for T_1 above:

$$- H_1 + T_1 \cos. K_1 = 0$$

in which, $\cos. K_1 = \frac{h_1}{R}$

However (see Fig. 10, plan, and Fig. 12, plan of foot ring), since

$$H_1 = 2\ P_1 \text{ sin. } e, \text{ and } e = \frac{360°}{2\ N} = \frac{180°}{N},$$

it is found that

$$P_1 = + \frac{1}{2 \text{ sin. } e} L_1 \text{ cotan. } K_1$$

in which, cotan. $K_1 = \frac{h^1}{r_1}$, and sin. $e = \frac{b}{2\ R_o}$

$$P_1 = + L_1 \frac{h_1\ R_o}{r_1\ b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

Panel Point II.—In Fig. 11, with reference to the values, T, Equations 31 to 34,

$$H_2 - T_1 \cos.\ K_1 + T \cos.\ K_2 = 0,$$

but sin. $e = \frac{b}{2\ R_o}$, cos. $K_2 = \frac{h_2}{R}$, and cotan. $K_2 = \frac{h_2}{r_2}$

$$P_2 = \frac{1}{2 \text{ sin. } e} \left[(L_1 + L_2) \text{ cotan. } K_2 - L_1 \text{ cotan. } K_1 \right]$$

$$= \frac{R_o}{b} \left[(L_1 + L_2) \frac{h_2}{r_2} - L_1 \frac{h_1}{r_1} \right] \ldots\ldots\ldots\ldots\ldots (36)$$

Panel Point III.—Etc.

$$P_3 = \frac{R_o}{b} \left[(L_1 + L_2 + L_3) \frac{h_3}{r_3} - (L_1 + L_2) \frac{h_2}{r_2} \right] \ldots\ldots\ldots\ldots (37)$$

Foot Ring.—See Fig. 13.

$$P_m = - \frac{R_o}{b} \left[(L_1 + L_2 + L_3 + \ldots + L_{m-1}) \frac{h_{m-1}}{r_{m-1}} \right] \ldots\ldots (38)$$

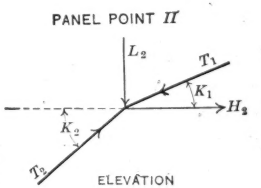


FIG. 11.

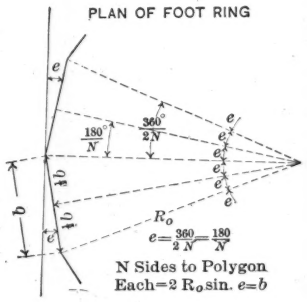


FIG. 12.

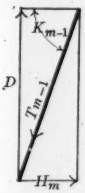


FIG. 13.

Graphic Method.

Dead Load.

Referring to Fig. 14, upper part, lay off from O to IV, the loads L_1, L_2, L_3, L_4, . . . , and form the force polygons for each panel point, as I, II, III, . . . , M, as follows:

Panel Point I.

Known: $L_1 = O\ I$, Unknown: T_1 and H_1,

Polygon: $O\ I\ a\ O$, now $T_1 = I\ a$, compression, $H_1 = a\ O$ (←)

Panel Point II.

Known: $L_2 = I\ II$, and $T_1 = Ia$, Unknown: T_2 and H_2,

Polygon: $a\ I\ II\ a$, now $T_2 = II\ b$, compression, $H_2 = b\ a$ (→)

Panel Point III.

Known: $L_3 = II\ III$, and $T_2 = IIb$ Unknown: T_3 and H_3,

Polygon: $b\ II\ III\ b$, now $T_3 = III\ c$, compression, $H_3 = c\ b$ (→)

Likewise: $T_4 = IV\ d$, compression, $H_4 = d\ c$ (→)

Panel Point V. (M).

Known: $D = L_1 + L_2 + L_3 + L_4$, and T_4 Unknown: H_m,

Polygon: $O\ IV\ d\ O$, now $H_m = O\ d$ (→)

The ring stresses can now be found by resolving them from their respective resultants, H_1, H_2, H_3, H_4, H_m, as shown in the upper part of Fig. 14.

The exact nature of the stresses is determined by knowing that the crown ring is always in compression, and that this is indicated by the plus sign. In the graphic method, it should be remembered that any force acting toward a panel point indicates compression, while the opposite action indicates tension. Further, that, in the triangle of forces, the resolvents have their arrows, indicating the sense of the force, running in the opposite direction to the arrow of the resultant. Proper care is to be taken to transpose these arrows from the force polygon to the respective members of the framework and *vice versa*. Thus, in the example, it is found that all the rings, except the crown ring, are in tension.

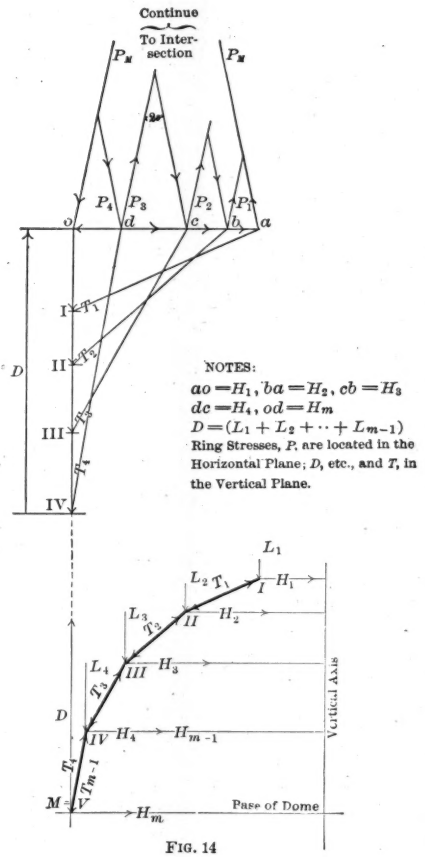


FIG. 14

Live Loads.

Let Q_1, Q_2, Q_3, Q_4, etc., designate the combined vertical components of the wind and snow loads on the respective panel points, then the maximum compression, and maximum tension stresses, which may possibly take place in the rings, due to these loads, will be as shown in Table No. 2.

TABLE No. 2.

	Maximum compression stress.	Maximum tension stress.	
Crown Ring.	$P_1 = Q_1 \frac{\text{cotan. } K_1}{2 \text{ sin. } e}$ $P_1 = + Q_1 \frac{h_1 R_0}{r_1 b}$	$P_1 = - 0.$	.(39)
Ring II.....	$P_2 = + Q_2 \frac{\text{cotan. } K_2}{2 \text{ sin. } e}$ $P_2 = + Q_2 \frac{h_2 R_0}{r_2 b}$	$P_2 = - \frac{Q_1}{2 \text{ sin. } e}$ (cotan. K_1 — cotan. K_2) $= - Q_1 \frac{R_0}{b}\left(\frac{h_1}{r_1} - \frac{h_2}{r_2}\right)$	(40)*
Ring III....	$P_3 = + Q_3 \frac{h_3 R_0}{r_3 b}$	$P_3 = - \frac{(Q_1 + Q_2)}{2 \text{ sin. } e}$ (cotan. K_2 — cotan. K_3) = $- (Q_1 + Q_2) \frac{R_0}{b}\left(\frac{h_2}{r_2} - \frac{h_3}{r_3}\right)$	.(41)
Ring IV....	$P_4 = + Q_4 \frac{h_4 R_0}{r_4 b}$	$P_4 = - (Q_1 + Q_2 + Q_3) \frac{R_0}{b}\left(\frac{h_3}{r_3} - \frac{h_4}{r_4}\right)$	.(42)
Foot Ring..	$P_5 = 0$	$P_m = - (Q_1 + Q_2 + \ldots + Q_{m-1}) \frac{R_0 h_{m-1}}{b r_{m-1}}$	.(43)

* Compare Equations 36, etc.

These results must be combined with the stresses found previously for the dead load.

For the rib stresses and the top and bottom rings, no separate formulas are needed, as a maximum load affects them most. It will be best to include the live loads at once with the loads, "L," in both the analytic and graphic determination of the stresses in these members.

Diagonals have been shown in all the panels of Fig. 8. For the dead load, none are required. For the effect of the live load, Schwedler proposed the following approximate solution:

Of two adjacent ribs, consider one only to be loaded with the loads, Q_1, Q_2, etc. Determine the rib stresses for each panel, and, for the several panels, assume this to be the shearing force acting on one of the diagonals in a panel. The other diagonal will take up a like amount of tensile stress.

Let d_1, d_2, d_3 . . ., be the angles formed by the diagonals with the rib-panel members, and D_1, D_2, D_3, . . . etc., the stresses in the diagonals; then, if F be the length of a rib panel, and l the length of a diagonal, approximately, cos. $d_1 = \frac{F_1}{l_1}$, cos. $d_2 = \frac{F_2}{l_2}$, etc., and

$$D_1 - = \frac{Q_1}{\text{sin. } K_1 \text{ cos. } d_1}, \text{ or } D_1 = - Q_1 \frac{R\, l_1}{r_1 F_1} \quad \ldots\ldots\ldots\ldots (44)^*$$

$$D_2 = - \frac{Q_1 + Q_2}{\text{sin. } K_2 \text{ cos. } d_2}, \text{ or } D_2 = - (Q_1 + Q_2) \frac{R\, l_2}{r_2 F_2} \quad \ldots\ldots\ldots (45)^*$$

Similarly,

$$D_3 = - (Q_1 + Q_2 + Q_3) \frac{R}{r_3} \frac{l_3}{F_3}, \text{ etc} \ldots\ldots\ldots\ldots\ldots (46)^*$$

By the graphic method, the diagonal stresses are found by developing a dome sector with its panels. With the known rib-panel stresses, $\Sigma\, Q_m \frac{R}{r_m}$, triangles of force can be formed for each panel point.

The fundamental formulas derived for the spherical dome with a homogeneous shell, of uniform thickness, hold good for domes of any other shape and of variable thickness, provided the proper substitution of the values for p and W be made therein.

In this it should be remembered that, in the expression, $p\, h$, p signifies the weight per square

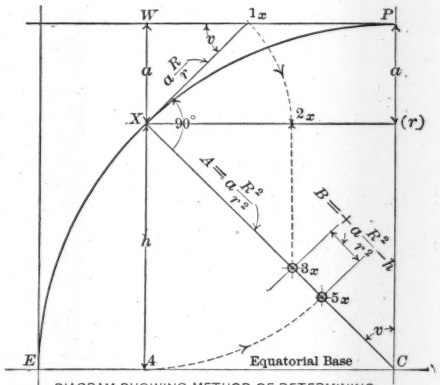


DIAGRAM SHOWING METHOD OF DETERMINING THE MAGNITUDE OF THE PRINCIPAL STRESSES FOR ANY POINT, X, OF A FULL DOME.

FIG. 15

* Derived, partly, by substituting in Equations 31 to 34, Q for L.

unit of shell, and for that part of it, only, which is located just below a particular parallel of latitude; whereas W represents the weight of the dome sector above that parallel. W, therefore, is not related to the now variable weight, p, in the same manner as it is in the case of the spherical dome of uniform thickness.

Concerning the Schwedler dome, those desiring more information are referred to existing literature, wherein the framed dome is treated as a statical indetermined structure, and wherein inclined loads are also considered. The writer believes that Schwedler's static method, as given in this paper, is accurate enough for all practical purposes.

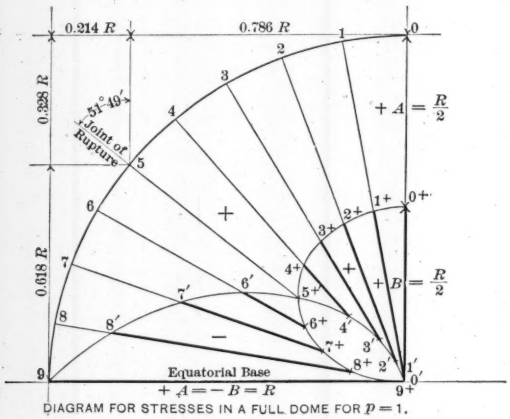


FIG. 16

For instance.

A_7 = Line 7 to 7^+, A_2 = Line 2 to 2^+

B_7 = Line 7^1 to 7^+, B_2 = Line 2^1 to 2^+

A_5 = Line 5 to 5^{+1}, $B_5 = 5^{+1} = 0$.

Below the Joint of Rupture:

Measure the A stresses over the extent of the light and heavy radi-ordinates.

Above the joint of rupture:

Measure the A stresses over the extent of the light lines only.

The B stresses are shown by the heavy lines.

Above the Joint of Rupture, B is compression.

Below the Joint of Rupture, B is tension.

Setting down again the expressions for the principal stresses for a spherical dome of uniform thickness as:

In general:

$$A = + W\frac{R}{r}, \qquad B = - W\frac{R}{r} + p\,h \ldots\ldots\ldots (47)$$

Spherical dome:

$$A = + \frac{R^2}{r^2} a\,p, \qquad B = - \frac{R^2}{r^2} a\,p + p\,h \ldots\ldots\ldots (48)$$

They can be readily produced graphically, as shown in Figs. 15 to 19. The initial quantity used in the construction of all the diagrams is:

$$W = R\,a\,p \ldots\ldots\ldots (49)$$

namely the weight of a zone-sector having a height equal to a, and a base equal to a unit of length at the equator.

This weight, W, is represented, for $p = 1$, in

Fig. 15, by the line, $W\,X = a$,
Fig. 17, " " " $W\,X' = a$.
Fig. 17, " " " $w\,X' = S_o$ for the zone S_o.

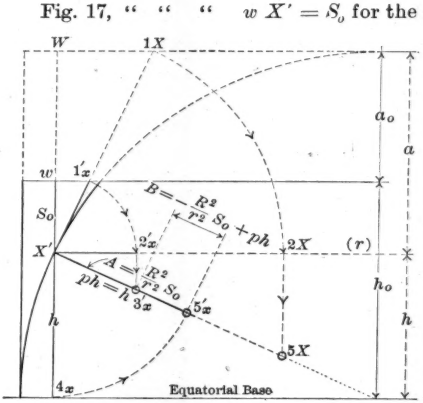


DIAGRAM SHOWING CONSTRUCTION OF THE MAGNITUDE OF THE A AND B STRESSES IN A TRUNCATED DOME FOR ANY POINT, X', ALSO FOR A FULL DOME AT THE SAME LATITUDE, FOR $p = 1$.

FIG. 17

In a truncated dome, $A = X'\,3'x$, $p\,h = X'\,5'x$, $B = 3'x\,5'x$.
In a full dome $A = X'\,5\,X$, $p\,h = X'\,5'X$, $B = 5'x\,5\,X$.

$$X'\,1'x = S_o\frac{R}{r}, \quad X'\,1X = a\frac{R}{r}.$$

See Equations 2, 3, 4 and 5.

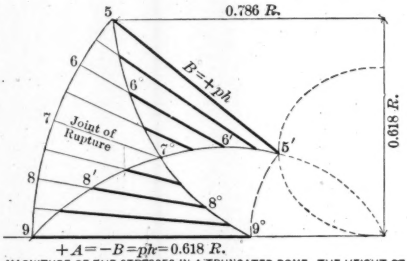
The construction is as follows:

Full Dome. Fig. 15.—For any point, X, and the radi-ordinate, CX, draw $W4$, produce XI_x at right angles to CX. Bring down, with X as a center, 1_x to 2_x, then vertically down to 3_x. Bring up to the same radius CX, with X as a center, the height, h, whence the stresses, A and B, have been laid off on the radi-ordinate, CX, as noted in the diagram.

Truncated Domes.—Proceed similarly.

Repeating this construction for a number of radi-ordinates, the diagrams, Figs. 16, 18 and 19 are obtained.

Examining Figs. 18 and 19, representing cases of truncated domes, it will be noticed that, as in the full dome, two zones exist, as regards the stresses, B; that is, there exists a special joint of rupture, above which B is compression, and below which is B tension, provided the base of the dome is at an elevation less than 0.618 R above the equatorial base.



$+A=-B=phc=0.618\ R.$

MAGNITUDE OF THE STRESSES IN A TRUNCATED DOME, THE HEIGHT OF WHICH HAS BEEN CHOSEN ARBITRARILY, AT 0.618 R, THAT IS, UP TO THE LEVEL OF THE PRINCIPAL JOINT OF RUPTURE OF A FULL DOME. SEE REMARKS RELATING TO FIG. 16.

B STRESSES ABOVE JOINT OF RUPTURE = COMPRESSION.
B STRESSES BELOW JOINT OF RUPTURE = TENSION.

FIG. 18

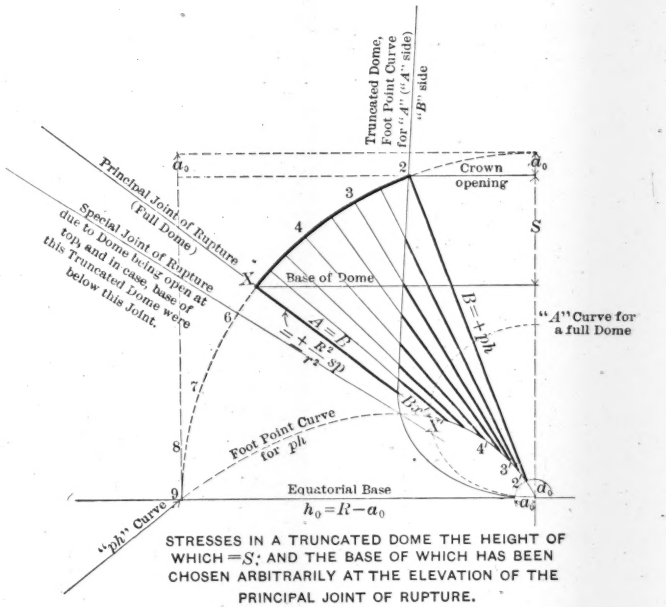
Stone-Masonry Coursing.—Entering now upon the question of how to design stone-masonry coursing that it may withstand tension, without regard to the frictional tenacity in the bed and the tensile strength of the mortar in the vertical joints, the writer will be guided by the well-recognized law that, in all important stone construction of a high character, no other material than stone should be used to connect the parts of the structure.

With this point in view, three methods of constructing the stone courses in the tension zones of masonry domes are suggested, as shown in Figs. 20 to 25.

The demands to be fulfilled by the different parts of each stone are as follows:

Each circumferential unit length of bed must be able to withstand a thrust, A.

The least cross-section must be able to withstand a tension, B.



STRESSES IN A TRUNCATED DOME THE HEIGHT OF WHICH $= S$; AND THE BASE OF WHICH HAS BEEN CHOSEN ARBITRARILY AT THE ELEVATION OF THE PRINCIPAL JOINT OF RUPTURE.

FIG. 19

In Fig. 19 note the large increase in the B stresses, in comparison with those occurring in a closed dome; also the corresponding reduction in the magnitude of the A stresses. The special point of rupture is determined by that radi-ordinate which passes through the intersection of the $p\,h$ and A curves.

A stresses are in light lines (except at the base).

B stresses are in heavy lines.

All stresses are compressive, except the B stress at the base, which is tension.

Note that, immediately below the base of the dome, p becomes zero, and that the stress represented by the line, $B\,x'$, is the compressive stress per unit length of meridian immediately above the bed-joint of the base.

If there be n stones in a course, or $2\,n$ tenons, each must be able to take up at its root a shear $= \frac{2\,r\,\pi\,B}{2\,n} = \frac{\pi}{n}\,(-W\,R + p\,h\,r)$. . . . (50)

Each tenon must also be able to take up, on the depth of its radial face, a like amount of compression.

Considering that, at the base of a dome, A and B have the same magnitude, and that the tensile strength of stones is much less than their compressive strength, it would seem to be sufficient to proportion the bottom course in all its parts for the stress, B; then all the other tension courses will be amply strong if similarly constructed. It may be, however, a great point in economy to lessen the depth of the tenons, from course to course, according to the decrease that takes place in the stress, B, from the base up to the joint of rupture. It is evident that, up to this joint, a stone of high tensile strength, and above this joint, a lighter material, of high compressive strength, should be used.

The lengths of the stones must be aliquot parts of the circumference. Use the radial thickness (bed) of the courses as the unit measure. Make the rise of the courses equal to the bed, and make the length of the stones as nearly as possible four times the rise.

For domes of variable thickness, a direct solution is not practical. The problem must be solved by successive trials, and for each parallel, if economy be an object.

Possibilities in Stone-Masonry Dome Construction.

Let it be required to find what diameter a half spherical dome of uniform thickness might have, if built with stones having:

Bed and rise = 1 ft.;

Weight = 164 lbs. per cubic foot;

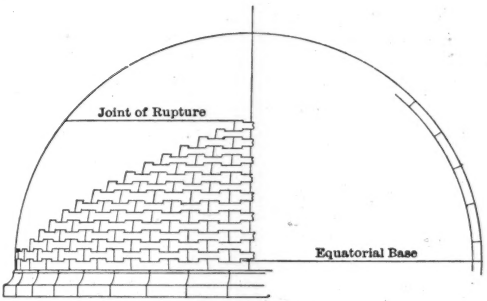
Safe tensile strength = 150 lbs. per square inch = 18 000 lbs. per square foot;

Safe compressive strength = 1 200 lbs. per square inch = 144 000 lbs. per square foot.

With regard to the latter stress, and Equation 17,

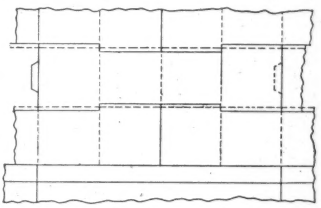
$$R = \frac{A}{p},$$

substitute, for A, 144 000 lbs., and, for p, 164 lbs., and find $R = 875$ ft., or diameter $= 2\,R = 1\,750$ ft.



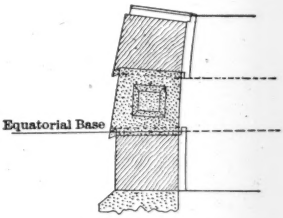
PART ELEVATION OF A DOME
DESIGNED WITH TENSION COURSES.

FIG. 20



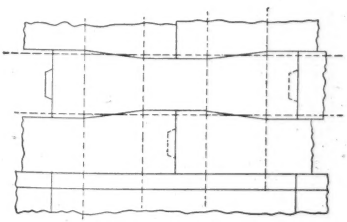
SHOULDERED COURSES.

FIG. 21



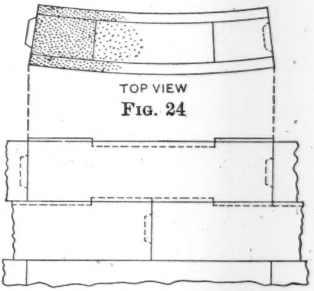
VERTICAL SECTION

FIG. 23



BEVELED STRETCHERS

FIG. 22



TOP VIEW

FIG. 24

CONCEALED BED AND BUILD TENONS

FIG. 25

With regard to the tensile strength of the stone, however, and Equation 17

$$R = \frac{B}{p},$$

wherein $B = 18\,000$ lbs., and p, again, $= 164$ lbs., it is found that $R = 110$ ft., say, or the diameter of the sphere $= 2\,R = 220$ ft. only.

Making the stones in the bottom course approximately 4 ft. long, the number of stones in each tension course would be:

(here for the base course) $n = \frac{2\,R\,\pi}{4} = \frac{R\,\pi}{2}$.

Each stone having two tenons, their number will be $= 2\,n$, or $= R\,\pi$.

For the compressive stress on the radial face of each tenon:

$$\frac{2\,R\,\pi\,B}{R\,\pi} = 2\,B,$$

now, the length of the tenons being 12 ins., and the safe permissible stress thereon $= 1\,200$ lbs. per square inch, their depth will be:

$$\frac{2 \times 18\,000}{12 \times 1\,200} = 2\tfrac{1}{2} \text{ ins.}$$

Of the two principal dimensions required for the execution of a dome, the first one usually selected is the radius of the shell. The second, the thickness of this shell, is the one which gives the designer the most worry. It is quite natural to assume this dimension to be a certain and definite fraction of the radius,* which, usually, is the first, or initial, dimension.

This, however, is not the case. The thickness of the shell of a dome is quite an arbitrary matter, if the dome has to carry only its own weight; but when the shell has to carry, besides, a lantern load or an additional extraneous weight, on its extrados, or intrados, then the thickness of the shell becomes at once a function of the radius and the external load.

Before proving the laws governing the principal dimensions of domes, they will be stated briefly here, and, as the writer believes, for the first time.

* This is the time-honored practice. The many existing domes have furnished the information by which these rules have been established and have found acceptance. In using such rules, the conscientious designer is aware of the absence of positive proof that he is doing right in following them.

1. The diameter at which domes, of uniform thickness, and carrying their own weight, may be executed, depends entirely on the tensile strength, or (if this be less), on the shearing strength of the material to be used in their construction.

2. For each kind of material, a "critical radius" obtains, beyond which the dome will not sustain itself, no matter how thick the shell may be.

3. Domes having a radius equal to, or within, the critical radius may be made, theoretically, as thin as it is possible to make them.

4. The critical radius of a dome is equal and equivalent to the length of a rod, of uniform cross-section, breaking by its own weight.

5. The factor of safety can only be secured by making the "working radius," at which the dome may be executed, as many times smaller than the critical radius as that factor indicates.

6. If this reduction, from the critical to the working radius, is made, then a surplus resisting strength per unit of cross-section (in the bottom course) is supplied, and may be utilized to take care of the stresses caused by other load than that of the shell of the dome. This surplus strength may be increased, within certain limits, at the will of the designer, as it depends entirely on the thickness which may be given to the shell.

7. In examining into the safety, and stability, of any existing dome, it will be necessary to ascertain, by tests, the character of the materials used in its construction (that of the mortar, or the frictional strength which, possibly, may exist in the bottom course). Then the ratio of the actual radius of the dome to the critical radius, belonging to the weakest resistance, will indicate the safety at which the dome may be considered to have been executed.

8 The critical radius may be increased by conforming the meridional cross-section of the shell to that of a triangle, or the shape of bodies of constant normal unit stress. This last form, however, in extremes, might lead to such a dimension for the thickness of the shell, at the base of the dome, as to be impractical of execution.

9. There exist, further (see No. 2), as many classes of critical, radii as there are kinds and classes of domes, and as there are formulas expressing the principal stresses at the base of such domes.

The formula for the tensile stress at the base of a hemispherical dome of uniform thickness is:

$$B = -Rp,$$

wherein, $p = wt =$ the weight of a radial prism, having a base (in the extrados) equal to a square unit of surface, and a radial height equal to the thickness, t, of the shell.

The product, Rp, therefore, represents the weight of a prism having a length equal to the radius, R, and whereof each linear unit has a weight, p. In fact, it is nothing else than the weight of a spherical sector, with a base equal to a unit of length, at the equator.

For equilibrium, between the tensile stress and the tensile resistance of the material, in the bottom course, we must have:

$$B = -Rp = -tk,$$

wherein $k =$ the ultimate tensile strength of the material per unit of cross-section of the shell of the dome.

Since $p = wt$,

$$Rwt = tk.$$

In this equation, t cancels, and, finally, the expression for the equilibrium of the stresses becomes:

$$Rw = k, \text{ or, } R = \frac{k}{w} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

This value for R is analogous and equivalent to that found for the length of a rod of uniform cross-section breaking under the action of its own weight.

Let $R =$ the length of the rod, *i. e.*, a rod containing R units of length;

$w =$ the weight per unit of length, and per unit of cross-section;

$k =$ the ultimate tensile strength for the same area, and at the point of suspension.

Then, for rupture at that place, there are obtained the equations (51), found before. The length, or the radius, given thereby may be properly called the "critical length" of a rod, or the "critical radius" of a hemispherical dome of uniform thickness.

If the rod is considered to be composed of an infinite number of threads, none of the conditions are changed, none of the threads will break, however small their cross-section may be assumed to be, until their length reaches the critical limit.

Thus, in the dome, the shell may be conceived to be divided in as

many, and as thin, concentric shells, and likewise the bottom course may be conceived to be composed of as many, and of as thin, threads as we please, and yet for any, and all of them, the condition:

$$\frac{k}{w} = R,$$

will be fulfilled, whether the shell be thick or thin.

In Table No. 4, the critical radius, R_c, has been calculated for a number of different materials, and the one belonging to "brickwork in lime," transferred to Table No. 3, for the purpose of comparing it in a general way with the actual radius at which these domes have been executed.

With what has been said before, as to safety in domes of "uniform thickness and hemispherical in shape," the behavior of the shells of the dome of St. Peter's in Rome (which approach closely to the two conditions just mentioned as to form and thickness of shell) is explained.

The dimensions of this dome, including the effect of the lantern load, seem to exceed the limits of the critical radius, or have, at least, very nearly reached it.

If k be the ultimate strength of the material at the critical radius, R, then, for a shell with a working radius equal to r, the stress per unit of cross-section will be only,

$$k_i = k\,\frac{r}{R},$$

each unit of cross-section, however, being able to withstand a stress, k. The surplus strength, k_s, per unit of cross-section, therefore, will amount to:

$$k_s = k - k_i = k\left(1 - \frac{r}{R}\right).$$

From applied mechanics, the thickness of a sphere under a uniform pressure, q, per unit of surface, is:

$$t = \frac{q\,r}{2\,k_s}.$$

Substituting herein, for k_s, the available surplus strength per unit of cross-section, the thickness of the shell of a dome, which has to carry, besides its own weight, an additional extraneous load, q, per unit of surface, is found to be

$$t = \frac{q}{2\,k} \times \frac{R\,r}{R - r}. \quad \ldots\ldots\ldots\ldots\ldots\ldots (51a)$$

TABLE No. 3.

Name.	Place.	Material.	Interior diameter.	Shape.	Thickness.		Date when built.	Name of builder.	Approximate critical diameter.
					At base.	At top.			
Dome of the Pantheon.	Rome.	Brick.	142 ft. Opening at the top, 28½ ft. in diameter.	Half sphere. Single shell.	7½ ft. Buttress, 21 ft., including base.	4 ft.	B. C. 25.	Marcus Agrippa (Roman).	6 x 60 ft.
Santa Sophia.	Constantinople.	Brick below, Tufa above. (?)	115 ft. Closed at the top.	Half sphere. Single shell. Resting on a stone base.	2¾ ft. Over buttresses, 8 ft.	2 ft.	A. D. 532 to 537.	Anthemius of Thales, and Isidorus of Milet (both Greeks).	2 x 60 ft.
Santa Maria del Fiore.	Florence.	Brick.	139 ft. Opening at the top, 20 ft. in diameter.	Egg-form. Eight-sided. Double shell, with lantern.	Inner shell: 7 ft. Outer shell: 2½ ft. Base, 15 ft. thick.	7 ft. 2½ ft.	A. D. 1421 to 1434.	Brunelleschi (Italian).	
St. Peter's.	Rome.	Rubble and brick, on stone base.	139 ft. Opening at the top, 25 ft. in diameter.	Egg-shape. Circular in plan. Double shell,* with lantern.	Inner shell: 6 ft. Outer shell: 3 ft. Base, 10 ft. thick.	6 ft. 3 ft.	A. D. 1513 to 1563.	Michael Angelo (Italian).	2 x 60 ft.

* Both shells are traversed by ribs connecting them along meridians from bottom to top.

Considering the case of a conical-shaped rod, as to its critical length: Rupture at the point of suspension (if R be the length of the rod and d the diameter of the base of the suspended cone) would occur at

$$\frac{1}{3}\frac{d^2\,\pi}{4}\,R\,w = \frac{d^2\,\pi}{4}\,k, \text{ and now: } R = 3\,\frac{k}{w}, \quad \ldots\ldots\ (51b)$$

Applying this result to the spherical dome, as in the case of a rod of uniform cross-section, it would seem correct to say:

If the meridional cross-section of a shell be made to resemble a (curved) triangle, then the critical radius will be three times as large as that for domes of uniform thickness.

Extending similar reasoning for dome shells, to suspended rods with a constant normal unit stress, then the thickness of a dome at the equator will be:

$$t = t_o\, e^{\frac{r\,w}{k}} \quad \ldots\ldots\ (51c)$$

if t_o = the thickness of the shell at the pole;

r = the assumed radius; and

e = the basis of the natural logarithms.

These assumptions, however, require to be proved.

TABLE No. 4.—AVERAGE WEIGHTS AND STRENGTHS OF BUILDING STONES, ETC., FROM VARIOUS SOURCES; ALSO THE VALUES OF THE CRITICAL RADIUS, R_o, WHEN USED IN DOMICAL STRUCTURES OF UNIFORM THICKNESS.

Material.	Weight, in pounds, per cubic foot.	Ultimate Strength, in Pounds per Square Inch.					Critical radius, in feet.
		Compression.	Tension.		Shearing.	Rupture.	
			Ultimate.	Elastic limit.			
Granite	173	12 000	1 500	1 125	4 300	1 800	1 260
Marble	168	8 000	1 000	750	2 900	1 200	852
Limestone	151	7 200	900	675	2 600	1 100	864
Sandstone	154	9 600	1 200	900	3 400	1 440	1 122
Bluestone	160	11 000	1 375	1 050	4 000	2 300	1 260
Brickwork in lime	112	1 000	50	.....	.....	.....	60
Brickwork in cement	120	2 000	300	225	450	500	360
Concrete	150	1 500	425	320	600	500	408
Terra cotta, hollow	.....	5 000	(?)	.....	(?)	1 500	.....
Solid terra cotta	144	2 000	300	.....	125	90	90
Aluminum	166	.....	.....	6 500	.....	.....	5 760
Steel	489	.....	.....	.....	48 000	.....	14 400
STRESSES USED BY THE REINFORCED CONCRETE CONSTRUCTION COMPANIES.							
Special rock concrete	150	2 400	200	.....	690	.....	192

In contemplating the construction of a stone masonry dome, the material to be used ought to be tested as to its strength. Table No. 4 is submitted for general information only.

With the exception of the dome of the Pantheon, the construction of the other three domes is wrong in constructive design.

In the Santa Sophia, forty openings are arranged around the base; and in the case of the last two structures the "Rib Principle" is a main feature. Both these arrangements are out of place in the tension zone of a spherical brick-masonry dome.

While no defects exist in the dome of the Pantheon, the other domes have shown fissures and cracks, presumably all below the joint of rupture, and have had to be reinforced by iron girdles, at least in the case of St. Peter's, a number of times.

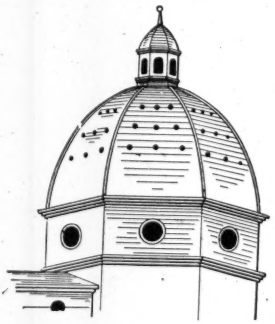
Theory of the Homogeneous Pyramoidal Dome, Containing All Other Forms as Special Cases.

In the first part of this paper only the spherical form of domes was considered. It can be shown, however, by making proper substitutions in the formulas of stresses for the pyramoidal dome to be derived, that the spherical, conoidal, conical, in fact, all forms of domes are special cases of this general case.

The ground plan of a pyramoidal dome, as implied by the geometrical name, may be any regular polygon.

Horizontal sections through this dome are all similar figures. The roof has usually a convex surface toward the outside. One of the two prime axial sections may be assumed to govern the design of the outline, which may be a circle, a circular arc, an ellipse, a parabola, etc. The use of a regular curve for the roof will enable the designer to produce and compute the various dimensions and weights of any or of the whole part of the surface with more ease than if the curve is an arbitrary one.

Fig. 26 gives an idea of the appearance of a pyramoidal dome. The "Duomo" of Florence, in Italy, is a representative example of this kind of dome, and is mentioned in the first part of this paper.



PYRAMOIDAL DOME.
Fig. 26

The theory of the stresses is based upon the following propositions:

I.—Any parallel of altitude (circumferential joint) divides the dome into an upper and a lower part.

II.—Each side of the polygonal bottom course (of the upper part) is assumed to act as a circumferential and connected tie.

III.—Each side of the polygonal top course (of the lower part) is assumed to be subjected to simple compression only.

IV.—The corners of all the polygonal horizontal courses are assumed to be (pin-) connected joints.

V.—The tensional circumferential stress in the bottom course of the upper part balances, by the tenacity of the homogeneous material of which the shell is constructed (and must therefore be equal to), the meridional downward, pressing and bursting thrust, due to the weight of the dome.

VI.—The tensional stress, mentioned under *V* is reduced, and may, at a particular altitude, be totally balanced, by the compressive stress existing in the top course of the lower part, and in case the separation, under *I*, ceases.

VII.—The weight of the dome, as it is transmitted downward from course to course, is assumed to be distributed uniformly around the circumference, at any altitude.

VIII.—No other (external) forces than its own weight act upon the dome.

In Fig. 27, let

H_0 = the sum of the forces acting upon each side of a polygonal course at any altitude;

P = the radial resultant of the H_0 forces, acting through the rib corners;

B = the tensional or compressional ring stress created in any course, in turn, by the radial forces, P.

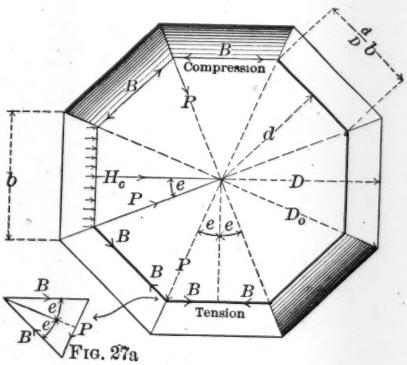


FIG. 27a

Number of Sides of Base $=N$

Circumference " " $=Nb$

$\sin. e = \frac{b}{\pi D_0}$

FIG. 27

Then, with reference to the dimensions and angles noted at the left side of the figure:

$$P = 2 \times \frac{1}{2} \frac{H_o}{\cos.\ e} = H_o \frac{D_o}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (52)$$

Forming (Fig. 27*a*) the triangle of forces, *B P B*, for any of the corners, gives:

$$2\ B \sin.\ e = P, \text{ or } B = \frac{P}{2 \sin.\ e} P = \frac{D_o}{b} \quad \ldots\ldots\ldots\ldots (53)$$

and, substituting for P the value of Equation 52, finally,

$$B = H_o \frac{D_o}{D\, 2 \sin.\ e} = H_o \frac{D_o^2}{D\, b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54)$$

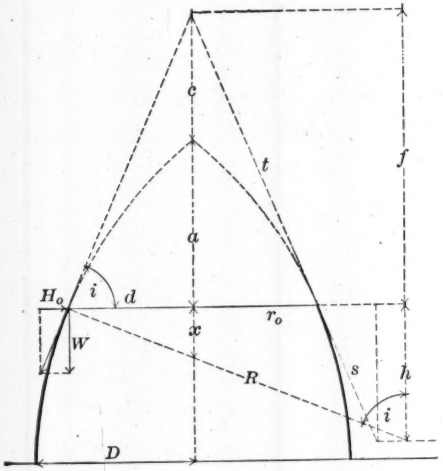


FIG. 28

In Fig. 28, D = Inscribed radius;

D_o = Circumscribed radius;

b = Length of each of the N sides at the base;

d = Inscribed radius at any parallel of altitude·

$x : d :: h : r_o\ x = \dfrac{d\,h}{r_o}$, $a + c = f$;

$x : d :: d : f,\ x = \dfrac{d^2}{f}$;

R = Radius of curvature at altitude, d;

$R^2 = r_o^2 + h^2$, $s : h :: t : (a + c) = f$.

For the magnitude of the compressive force at the top course of the lower part, in Fig. 28:

Let W = the weight of each side of the top course, the rise of which, along the surface of the dome, being equal to unity;

R = the radius of curvature of the dome surface along the meridian and for the unit length thereof, from the top edge of the same course downward;

h = the vertical distance of the evolute point of R below the top course;

D = the short diameter, at the base of the dome;

d = " " " " " top of the dome;

H_o = the horizontal inward thrust for each side of the top course, created by the weight, W, of the same course.

Then, by forming the triangle of forces with H_o and W, and comparing the same with the similar triangle formed by the dimensions of the dome, R, r_o, and h, the result is the proportion:

$$H_o : W = h : r_o,$$

or,
$$H_o = W\frac{h}{r_o}, \text{ or, } H_o = W\frac{x}{d} \quad \ldots\ldots\ldots(55)$$

The forces at the base of an upper part of the dome, say, for instance, at an elevation, d, are found, with reference to Proposition V, and Fig. 29, as follows:

Let W = the total weight of the dome above the parallel, d;

A = the resulting tangential meridional thrust;

R = the radius of curvature of the dome surface at the same altitude.

Then, by similar triangles:

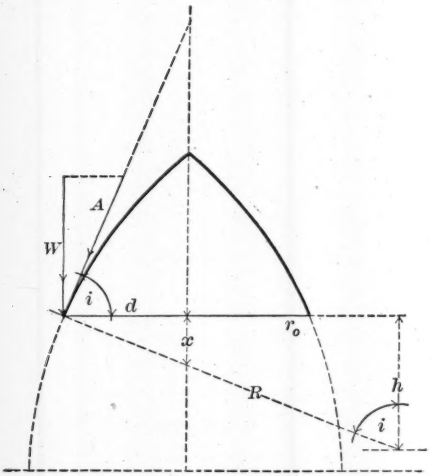
$$A : W :: R : r_o, \text{ or } :: s : h,$$

whence
$$A = + W\frac{R}{r_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(56)$$

and also
$$A = + W\frac{s}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(57)$$

For equilibrium around the polygonal base, the sum total of the circumferential tensile stresses, B, must be equal in amount to A, but opposite in character, and will therefore be:

$$B = - W\frac{R}{r_o}, \text{ or } B = - W\frac{S}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots (58)$$



$A : W :: R : r_o$

$A = W\frac{R}{r_o}$

FIG. 29

Formulas 55 to 58 give the gross amount of the magnitude of the several stresses in their most general form, and for the gross weights of the dome shell.

In the following formulas the stresses will all be reduced to "per unit length," and "per unit of cross-section" of the shell of the dome.

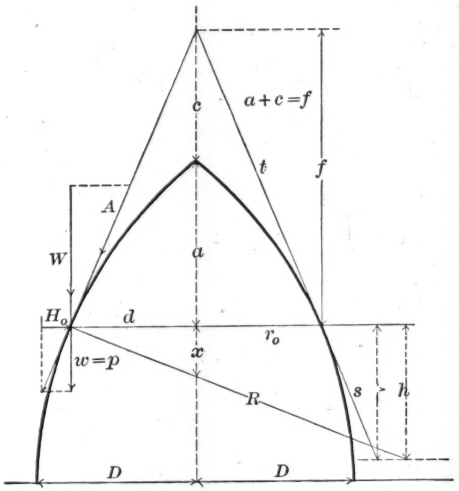
Referring, further, all dimensions and weights at any altitude to the dimensions and weights of the base course, gives the following relations (see Figs. 27 and 30):

Compressive Ring Stress in the Top Course of the Lower Part.—Let $p\,b$ = the weight of one side of the main base course,

$$\text{then } W = p\,b\,\frac{d}{D}$$

which equals the weight of one side of a polygonal course at the altitude, d. Substituting this value for W in Equation 55:

$$H_o = p\,b\,\frac{d\,h}{D\,r_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (59)$$



$w : H_o :: r_o : h :: d : x :: f : d.$
$W : A :: r_o : R :: f : t :: h : s.$
$r_o = R \sin. i, \qquad x = \frac{dh}{r_o} = \frac{d^2}{F}$

FIG. 30

this, in Equation (52), gives:

$$P = p\,b\,\frac{d\,h}{r_o}\,\frac{D_o}{D^2} \quad \ldots\ldots\ldots\ldots\ldots \quad (60)$$

and, finally, this value in Equation 53, the compressive stress per unit length at the top of an open dome will be:

$$B = +\,p\,\frac{d\,h}{r_o}\,\frac{D_o^2}{D^2}, \text{ or } B = +\,p\,x\,\frac{D_o^2}{D^2} \quad \ldots\ldots\ldots \quad (61)$$

For the two principal stresses at the base of any dome, we had:

$$A = +\,W\,\frac{R}{r_o}, \text{ and } B = -\,W\,\frac{R}{r_o} \quad \ldots\ldots\ldots \quad (62)$$

Let $N\,b$ = circumference of the base course, then

$N\,b\,\frac{d}{D}$ = the circumference of the dome at any altitude, as d,

and for the unit length of circumference:

$$A = +\,W\frac{R\,D}{r_o\,N\,b\,d}, \quad B = -\,W\,\frac{R\,D}{r_o\,N\,b\,d} \quad \ldots\ldots\ldots \quad (63)$$

By uniting the upper and lower parts of the dome again, having been conceived until now separated, under Proposition *J*, and com-

bining the equations derived for the stresses under that condition, the actual stresses existing at any point in the surface of a pyramoidal dome are found.

Combining, therefore, Equations 61 and 63, with reference to Fig. 30, the magnitude of the tangential meridional thrust, A, per unit of length of circumference, is found to be:

$$A = + W \frac{R\,D}{r_o\,N\,b\,d}, \text{ or, also, } A = + W \frac{s\,D}{h\,N\,b\,d} \quad \ldots\ldots\ldots (64)$$

For the shearing stress along the surface of the bed-joint of a course, and per unit of length of the same, and for the tensile stress in the radial cross-section of the same course, having a rise equal to a unit of length of meridian:

$$B = - W \frac{R\,D}{r_o\,N\,b\,d} + p \frac{d\,h}{r_o} \frac{D_o^2}{D^2} \quad \ldots\ldots\ldots\ldots\ldots (65)$$

$$\text{or, also, } B = - W \frac{s\,D}{h\,N\,b\,d} + p\,x \frac{D_o^2}{D^2} \quad \ldots\ldots\ldots\ldots\ldots (66)$$

These, then, are the general equations containing the theory of the principal stresses in domes of a pyramoidal shape.

To make good the statement that other forms of domes are only special cases of this form of dome, and for the reason that such specialization is the purpose of applied mechanics, the formulas for the stresses in spherical, conical, and conoidal-shaped domes will now be found.

The Spherical Dome.

The several kinds of this dome have already been treated in the first part of this paper, and, therefore, attention will be confined to the production of the formulas for the stresses in a full or hemispherical dome only.

Referring to Figs. 27 and 30, and Equations 64 and 65, the following substitutions will be made:

$N\,b = 2\,R\,\pi$, circumference of the dome at base;

$D_o = D = R$, radius of the hemisphere;

$d = r_o$, radius of any parallel of altitude;

$h = R - a$, vertical distance between the evolute point of R (*i. e.*, center of sphere) and the joint in question;

$W = 2\,R\,\pi\,a\,p$, weight of the whole spherical segment of the dome, above the same.

Then, after reducing:

$$A = + \frac{R^2}{r_o^2}\, a\, p \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (67)$$

$$B = - \frac{R^2}{r_o^2}\, a\, p + p\, h \ldots\ldots\ldots\ldots\ldots\ldots (68)$$

Considering that $a = R - h$, $r_o^2 = R^2 - h^2 = (R + h)(R - h)$,

then, also, $A = + \dfrac{R^2}{R + h}\, p$, and $B = - \dfrac{R^2}{R + h} + p\, h \ldots\ldots\ldots\ldots (69)$

These results are the same as given by Equations 14 and 15, in the first part of this paper. The conclusions arrived at there need not be repeated again.

The Conical Dome.

Referring to Equations 64 and 66, and Figs. 27, 30 and 31, and conceiving the necessary modifications to be made, to transform Fig. 30 into a conical dome, as shown in Fig. 31, we have first:
$c = 0$, $f = a$, $D_o = D$, $t = S$, $r_o = d = r$, $h = x$, and, as in Fig. 31,

$$x = \frac{r^2}{a}.$$

Recalling now, for a moment, the algebraic expression for the envelope of a cone, we have, further:
$W = r\, \pi\, S\, p$, total weight of the conical surface;

$W = \dfrac{r\, \pi\, S\, p}{2\, r\, \pi}$ = the weight of a cone sector thereof, with a base =

unit of length $= \dfrac{S}{2}\, p.$

This result is obtained, also, by reducing the expressions in Equation 63:

$$W \frac{D}{N\, b\, d} = \frac{W}{N\, b\, \dfrac{d}{D}} = \frac{r\, \pi\, S\, p}{2\, r\, \pi} = \frac{S}{2}\, p.$$

Conforming now the expressions for A and B (Equations 56, 57 and 58), to the quantities just found, they become:

$$A = + \frac{S^2}{2\, a}\, p \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (70)$$

$$B = - \frac{S^2}{2\, a}\, p_o + \frac{r^2}{a}\, p_u \ldots\ldots\ldots\ldots\ldots\ldots (71)$$

The magnitude of the two terms in the equation for B will determine, when evaluated, whether there is tension or compression in a

course. In its present shape, not much can be learned from the formula in this respect; but by the substitution,

$$S^2 = a^2 + r^2, \text{ and } p_o = p_u = p,$$

it can be brought to the form:

$$B = -p\left(\frac{a^2 - r^2}{2a}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (72)$$

Interpreting this formula, the following statements can be made:

If a equals r, then the cone has a slope of 45°, B becomes zero, that is, none of the courses is in tension; each simply transmits, from course to course downward, the thrust, A.

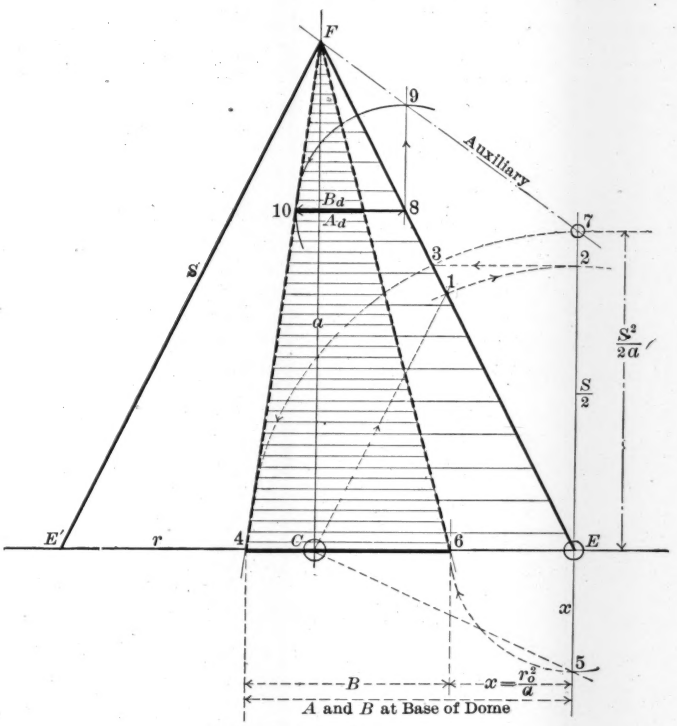
If, now, a be greater than r, the inclination of the sides of the cone to the horizon is more than 45°, and B will have the negative, or tension sign. All courses, therefore, from the top down, in a complete cone, sustain tension (and compression due to A).

If the inclination be less than 45°, the courses are subjected to compression in the direction of the parallels of altitude, as well as in the direction of the sloping sides of the cone.

For the determination of the tensional stress at the bottom of the main base course, this formula is not applicable, and, instead, the original form of the formula, Equation 71, must be used for this purpose.

Remembering that the first term therein relates to the effect of the upper part, and the second term to the effect of the lower part, of the dome, in reference to a particular altitude, it will be seen that at the base of any dome the second term vanishes.

A and B become there equal in amount, but are opposite in character.



HIGH CONICAL DOME.

FIG. 31

In Fig. 31, $E'\text{-}C = C\text{-}E = r$;

$C\text{-}1 \parallel E'\text{-}F$, $C\text{-}1 = 1\text{-}E = \frac{1}{2}\, E\text{-}F = \frac{S}{2}$;

$2\text{-}E = \frac{S}{2}$, $2\text{-}3 \parallel E'\text{-}E$;

$3\text{-}E : 2\text{-}E :: S : a$, $3\text{-}E = \frac{S^2}{2\,a}$;

$C\text{-}5 \perp C\text{-}1 = \perp E'\text{-}F$; $x : r = r : a$, $x = \frac{r^2}{a}$;

$A = \frac{S^2}{2\,a}$, $B = -\frac{S^2}{2\,a} + x$, $p = 1$ (unit).

$4\text{-}E = A$, $4\text{-}6 = B$, $6\text{-}E = x = \frac{r^2}{a}$,

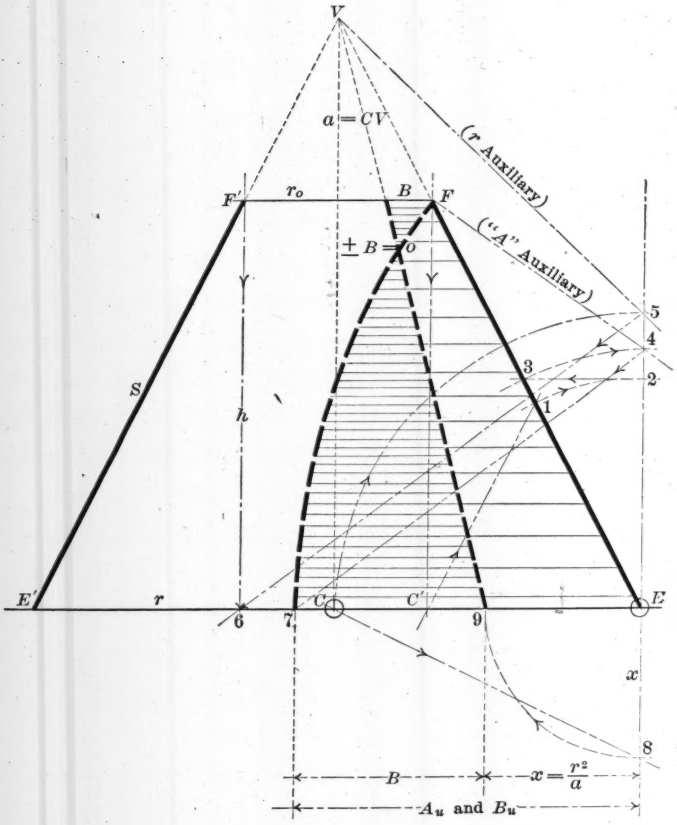
$E\text{-}F$ = Base line for the stresses A,

$6\text{-}F$ = " " " " " B,

4-F = Terminal line for both A and B; however, any other point of this line can be constructed by the use of the auxiliary line 7-F;

$$7\text{-}E = \frac{S^2}{2\,a} = 3\text{-}E.$$

For Point, 8, for instance, form the circuit 8-9-10.



TRUNCATED OR LOW CONICAL DOME.

FIG. 32

In Fig. 32, h = height of frustum;

S = length of side;

r_o = radius at top;

r = radius at any joint below top;

C'-1 $\parallel$ E'-F', I-$E = \frac{1}{2} S$, 2-3 $\parallel$ E'-E, 3-$E = \frac{S^2}{2\,h}$;

5-E = C-E = r, 6-C = r_o, 6-E = $r_o + r$, draw 5-6,

then, 4-7 $\parallel$ 5-6, now E-7 $= \frac{S^2}{2\,h} \times \frac{r + r_o}{r} = A$, C-8 $\perp$ E^1-F^1;

9-E = 8-E = $x = \frac{r^2}{a}$, 7-9 = $B = -\frac{S^2}{2\,h} \times \frac{r + r_o}{r} + \frac{r^2}{a}$;

E-F = base line for A;

9-V = base line for B;

Curve 7-F = terminal line for A and B.

The Truncated or Low Conical Dome.

Referring to Fig. 32, and the above-mentioned general formulas for A and B, we have, for the stresses per unit length of circumference at any parallel, first:

$W = S\,\pi\,p\,(r_o + r)$, the weight of the envelope from r to r_o, and

$W = \frac{S}{2}\,\frac{r_o + r}{r}\,p$, the weight of a cone sector thereof with a base = unity, at the parallel of altitude, r.

Introducing this latter value, in the formulas, and assuming for a moment that the line E' - E, be an intermediate point between the actual base of the dome and its top:

$$A = +\,p\,\frac{S^2}{2\,h}\,\frac{r_o + r}{r} \qquad \text{(73)}$$

$$B = -\,p\,\frac{S^2}{2\,h}\,\frac{r_o + r}{r} + \frac{r^2}{a}\,p \qquad \text{(74)}$$

The Truncated Dome with a Lantern Load.

To obtain the stresses, A and B, in this case, we have to add to the weight, W, in the above formulas, the effect of the lantern load, Q, at the parallel, r, in question.

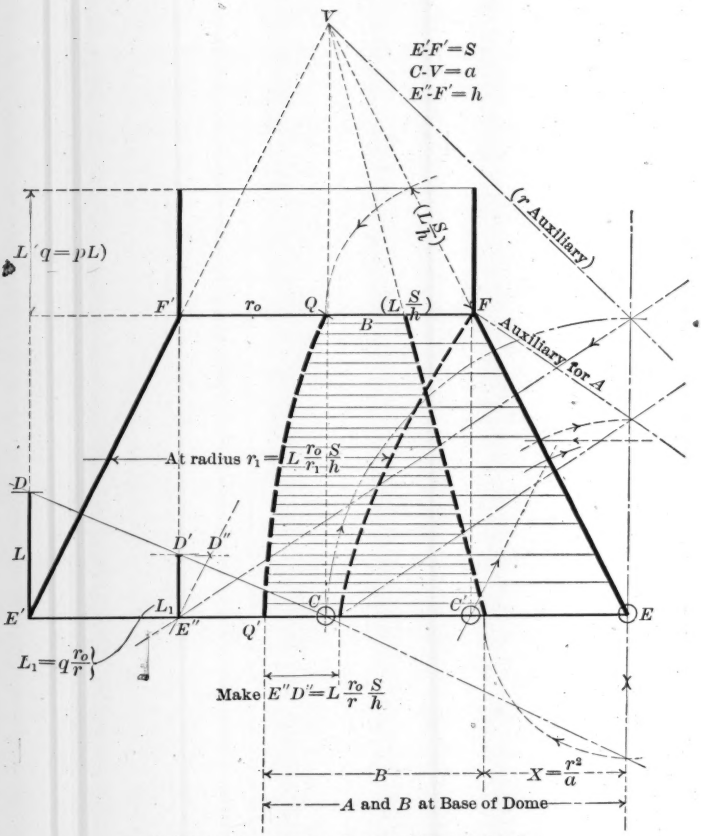
If $q = \frac{Q}{2\,r_o\,\pi}$, the load per unit of circumference at the top, then

$Q = q\,2\,r_o\,\pi\,p$, and

$W = q_r = \frac{Q}{2\,r\,\pi} = q\,\frac{r_o}{r}$, the load per unit of circumference at the parallel, r. Now:

$$A = + q \frac{r_o}{r} \frac{S}{h} + p \frac{S^2}{2h} \frac{r_o + r}{r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (75)$$

$$B = - q \frac{r_o}{r} \frac{S}{h} - p \frac{S^2}{2h} \frac{r_o + r}{r} + \frac{r^2}{a} p \quad \ldots\ldots\ldots\ldots\ldots\ldots (76)$$



LOW CONICAL DOME WITH LANTERN LOAD.

FIG. 33

In Fig. 33, L = height of superimposed load, assumed to have a weight = p per square unit; then weight on top = $p \times L$.

Make E'-$D = L$, draw D-C, E''-$C = r_o$, E''-$D'' \parallel E'$-F'; then, as noted for the lower joint, E''-$D'' = p\, L \frac{r_o}{r} \frac{S}{h}$.

Now find:

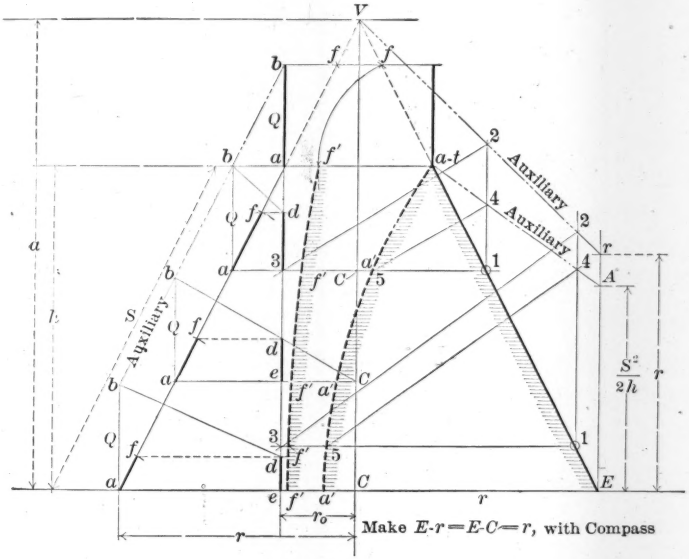
$$A = + p\,L\,\frac{r_o}{r}\,\frac{S}{h} + p\,\frac{S^2}{2\,h}\,\frac{r + r_o}{r} \text{ (see Fig. 32),}$$

$$B = - p\,L\,\frac{r_o}{r}\,\frac{S}{h} - p\,\frac{S^2}{2\,h}\,\frac{r + r_o}{r} + p\,\frac{r^2}{a}\,.$$

Stresses are laid off for p = unit.

For further explanations see Figs. 31 and 32.

All the expressions for the stresses, A and B, as given above, can be readily constructed, as shown, and fully explained, in Figs. 31, 32, and 33. Some of the lines therein are designated as "auxiliaries." Examining Fig. 34, the manner in which these auxiliary lines assist and simplify the construction of the stresses will become manifest.



CONSTRUCTION OF THE STRESS "A," IN A LOW DOME WITH A LANTERN LOAD, Q, AND FOR A WEIGHT = p, PER SQUARE UNIT OF SURFACE.

FIG. 34.

In Fig. 34, construct $E\text{-}A = \frac{S^2}{2\,h}$, as previously shown, and $E\text{-}r = r$. Draw the auxiliary lines, $A\text{-}a\text{-}t$ and $r\text{-}V$; and $b\text{-}b\text{-}b$ . . . for Q.

For any point, as 1, form the circuits, 1-2-3-1, and 1-4-5-1, 4-5 || 3-2; then 1-5 = A, for the cone only.

Form, also, the circuits a-b-C, e-d-f-a; make a'-f' = a-f; a-f = the stress, A, for $Q = q \frac{r_o}{r} \frac{S}{h}$, and 1-$f'$ = A, total.

Line a-t-E = base line, f'-f' . . . = terminal line for the A stresses.

Conoidal or Pointed Domes.

The conoidal domes resemble the pyramoidal domes in their axial sections; they are, however, circular in plan.

If W be the weight of the dome above a particular parallel of altitude, and 2 d π the circumference of that circle, then the stresses per circumferential linear unit become (Fig. 30):

$$A = + \frac{W}{2\,d\,\pi} \frac{R}{r_o} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (77)$$

$$B = - \frac{W}{2\,d\,\pi} \frac{R}{r_o} + p \frac{d\,h}{r_o} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (78)$$

Joints of Rupture in Conical Domes.*

Examining the four diagrams showing graphically the magnitude of the three prime forces acting at an intermediate point of the shell, one located between the top and the base of a conical dome, it will be noticed that in a full dome, as Fig. 31, no joint of rupture exists. The ring stresses will be either tension, zero, or compression, from top to bottom, according to the inclination the sides of the dome have to the horizon, and as previously explained.

In the truncated dome, however, such a joint exists (see the point marked $\pm B = 0$, in Fig. 32). Above this joint the ring stresses are compression, while below it, they are tension. A similar point, $\pm B = 0$, will be found in Fig. 33. The effect of the lantern load, however, overbalances, in this instance, the compressive ring stress in the compression zone, and all the courses are subject to tension, from the base of the lantern down to the base of the dome.

In conclusion, some stable forms of domes are submitted, in which there is no tension in any direction, except the tensional shear in the bed-joint of the lowest course. In each instance it will be necessary

*Rather say, "Joints of Equilibrium."

FORMS OF STABLE CIRCULAR DOMES OF MASONRY.

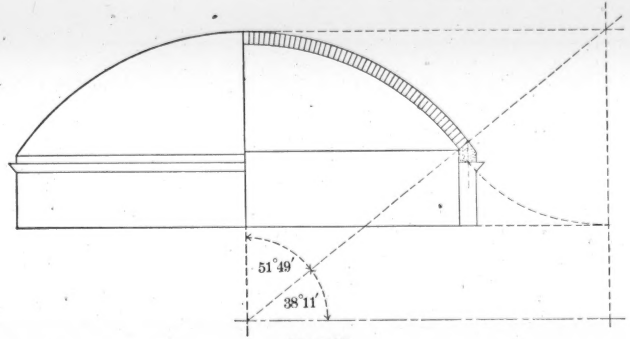


FIG. 35

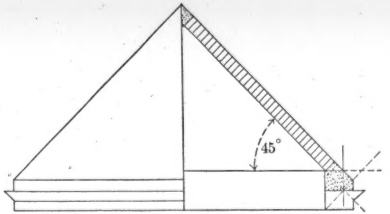


FIG. 37

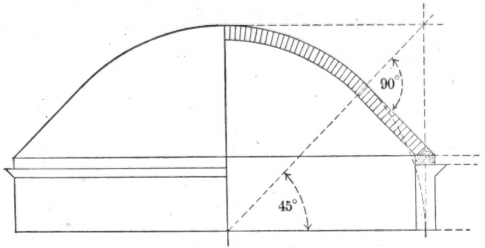


FIG. 36

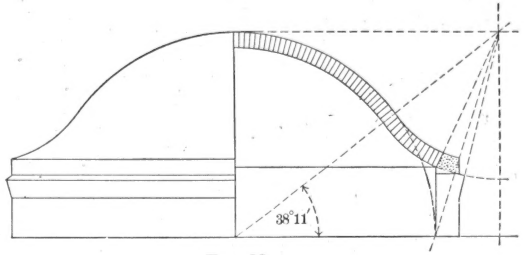


FIG. 38

to resist the radiating pressure in this joint, and there only, by the tenacity of a hoop.

In masonry domes with a tension zone, the whole of this zone should be encased and covered by a sheet of metal, able to take up the tension stresses, B, in case the construction of tension courses, suggested previously, should prove too expensive.

In buildings of a monumental character, where money is of secondary importance, the designer should, perhaps preferably, use other means than the ones just mentioned to counteract the stresses in the tension zone. A ring of heavy masonry should encircle the dome from the "joint of equilibrium" down to the base. The dimensions of this wall should conform amply to the ring stresses, B, mentioned throughout this paper.

Reviewing past practices, in this respect, it is found that in only one of the famous ancient domes has the designer carried out this principle effectively and completely, namely, in that of the dome of the Pantheon, in Rome.

DISCUSSION.

Mr. Church.

IRVING P. CHURCH, ASSOC. AM. SOC. C. E. (by letter).—The writer will confine his discussion (except as below) to the first few pages of the paper, involving a demonstration of the stresses in the ordinary spherical dome; which is understood to be homogeneous, of uniform small thickness, and composed of small blocks or voussoirs the surfaces of contact of which are vertical meridian planes on the sides, and conical beds at the top and bottom, each such conical surface being part of the surface of a right cone having a vertical axis and its vertex at the center of the sphere. Since these bearing surfaces are assumed to be smooth, the stress, if compressive, is at right angles to the surface; while, if the stress is tensile (as may occur on the sides), it will be supposed that a horizontal ring of blocks is continuous in substance, so as to keep its form and position.

While the final general results, stated in Equation 21, for the case of the open dome (and this includes all the other cases), are correct, the manner of demonstration, in the writer's opinion, is extremely obscure and elusive. The first obscurity occurs in Case II on page 265. Here, the author states: "At the base of any dome, two principal stresses exist, and must balance each other." This might be taken as the common loose way of saying "certain forces balance each other," when what is really meant is that a certain body, under the action of a certain set of forces, is in equilibrium, or "balanced," under those forces; but, at the foot of the page it is plainly implied that these "principal stresses," per unit length of their respective beds or surfaces, are assumed equal to each other. The author does not describe, nor represent by diagram, any special body or voussoir on which both these forces act, so as to prove their equality by the laws of mechanics, but, apparently, makes the assumption of equality without adequate reason; that is, that $A = B$. The proof that the meridional thrust, A, per unit length of horizontal circumference, is equal to $\frac{R^2}{r^2_u} a\, p$, is sound; but that B should be equal to A would appear to be pure assumption, no valid reason being adduced, that the writer is able to discover. The obscurity is deepened by the statement that B is the force "per unit length of circumference, and for one unit length along the meridian." If B is the "horizontal ring stress" it is difficult to see how it can be measured per unit length of (horizontal) circumference. Perhaps the author might have made himself better understood if this force, B, had been shown, in a diagram or figure, as acting on some definite body.

As the result of an endeavor to gain some insight into what the author means by the force, B, of Case II, the writer offers the following

Mr. Church. treatment of the spherical dome, in which the use of trigonometric functions (except in one instance), and of the notation of the differential calculus, have been avoided purposely.

In order that results may apply to any spherical dome, the case of the open dome has been taken; supposed to be divided up regularly (by meridional planes and conical surfaces), into small voussoirs.

The notation used is the same as in the paper (with other symbols added as needed). In Fig. 39, $M\,N$ is a typical small voussoir situated in a ring, the radius drawn to the upper edge of which makes an angle, α, with the vertical axis of the dome, while that to the lower edge

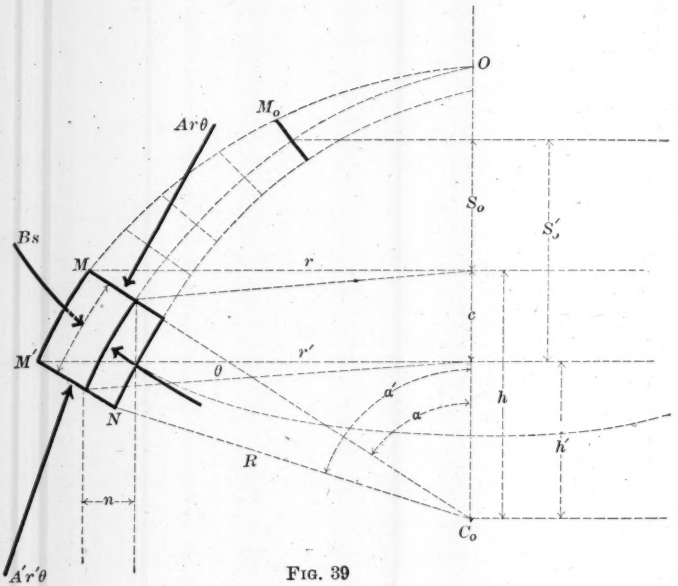


FIG. 39

makes an angle, α'. A voussoir of the crown ring of this open dome is shown at M_o, in the same "ungula" as $M\,N$. R is the radius of the sphere, while r and r' are the radii of the horizontal circles passing through M and M'. The ungula, $O\,M'\,N$, subtends a small angle, θ, at the vertical axis. Let the small length, $M\,M'$, be denoted by s, its vertical projection being c, and its horizontal, n. C_o is the center of the sphere. The system of forces acting on the voussoir, $M\,N$, and holding it in equilibrium, consists of the two meridional thrusts from neighboring voussoirs above and below, two lateral thrusts from the adjoining voussoirs on the sides, and the weight of the voussoir itself. This

weight is not shown in the figure; it has no horizontal component and hence does not affect the mathematical work (based on Fig. 40) which is to follow. Each of the thrusts is at right angles to the corresponding surface, the two lateral ones being, of course, equal in value. Mr. Church.

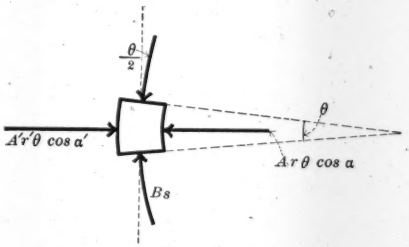


FIG. 40

Since A is to denote the meridional thrust per unit length of horizontal circumference, while B is the side thrust or "hoop thrust" (perhaps tension) per unit length of meridian, the values of the thrusts will be, respectively, $A\,r\,\theta$, $A'\,r'\,\theta$, and $B\,s$; as marked in Fig. 39. By the same process as that pursued on pages 265 and 266 of the paper, it may easily be proved that

$A = \frac{R^2}{r^2} S_o p$; and $A' = \frac{R^2}{r'^2} S'_o p$, (in which $p =$ the weight of the dome per unit of spherical surface); while the value of B is to be determined.

Fig. 40 shows the horizontal projection of the system of forces acting on the block or voussoir, $M\,N$, from which it is seen that, since the block is in equilibrium, the sum of the radial components (*i. e.*, parallel to the line bisecting the small angle, θ) of the two forces, $B\,s$, must be numerically equal to the difference of the two radial forces involving A and A'. If this fact is formulated and the above values of A and A' inserted, and, also, if $\frac{h}{R}$ is written for cos. α, and $\frac{h'}{R}$ for cos. α', there results $R^2\,p\,\theta\left[\frac{S'_o}{r'}\frac{h'}{R} - \frac{S_o}{r}\frac{h}{R}\right] = 2\,B\,s\,\frac{\theta}{2}$; and,

further, $B = p\,\frac{R}{s}\left[\frac{S'_o}{r'}\,h' - \frac{S_o\,h}{r}\right]$; .....(79)

which,

since $S'_o = S_o + c$, $h' = h - c$, and $r' = r + n$, or $r' = r\left[1 + \frac{n}{r}\right]$,

may be written in the form, $B = \frac{p\,R}{s\,r}\left[\frac{(S_o + c)\,(h - c)}{1 + \frac{n}{r}} - S_o\,h\right]$.

But, for an indefinitely small value of n, $\frac{1}{1 + \frac{n}{r}}$ may be replaced by $1 - \frac{n}{r}$, whence $B = \frac{p}{s}\,\frac{R}{r}\left[\left(1 - \frac{n}{r}\right)(S_o + c)\,(h - c) - S_o\,h\right]$ ....(80)

Mr. Church.

or, $B = \frac{p}{s}\frac{R}{r}\left[S_o h - S_o c + c h - c^2 - \frac{S_o h}{r} n + \frac{S_o c}{r} n - \frac{h}{r} n c + \frac{n c^2}{r} - S_o h \right]$ ..... (81)

After cancelling the terms, $S_o h$ and $-S_o h$, it is noted that, of the terms remaining in the bracket, all but three contain as factors two or more of the small lengths, c or n, while the three in question contain only one; hence as n and c become indefinitely small (and finally zero, since it is wished to discover the value of B for a mere point close to the upper corner of the block) the former terms will vanish, as compared with the three;

whence, $B = \frac{p R}{r}\left[\frac{c}{s} h - S_o \left(\frac{c}{s} + \frac{n}{s}\frac{h}{r}\right)\right]$.

But, from similar triangles, $\frac{c}{s} = \frac{r}{R}$ nd $\frac{n}{s} = \frac{h}{R}$;

and, therefore,

$$B = p\left[h - S_o\left(1 + \frac{h^2}{r^2}\right)\right] = p\left[h - S_o\left(\frac{r^2 + h^2}{r^2}\right)\right];$$

that is, $B = p\left[h - \frac{S_o R^2}{r^2}\right]$; or $B = -\frac{R^2}{r^2} S_o p + p h$ ..... (82)

which is identical with the result in Equation 21 of the paper. (The writer, however, sees no occasion for the reservation made by the author in the few lines immediately under Equation 21.)

Now note the effect of supposing the horizontal ring, to which the block, $M N$, belongs, to be destitute of weight, all the rings above it still retaining the property of weight. Then, while A remains as before, the value of A' becomes

$$A' = \frac{R^2}{r'^2} S_o p.$$

The same steps being followed as before, it is found that, instead of Equation 79, there is obtained

$$B = \frac{p R}{s}\left[\frac{S_o h'}{r'} - \frac{S_o h}{r}\right] = \frac{p R S_o}{s}\left[\frac{h'}{r'} - \frac{h}{r}\right] \text{ ..... } (79a)$$

and, instead of Equation 80,

$$B = \frac{p}{s}\frac{R}{r} S_o\left[\left(1 - \frac{n}{r}\right)(h - c) - h\right] \text{ ..... } (80a)$$

whence, $B = \frac{p}{s}\frac{R}{r} S_o \left[h - c - \frac{h}{r} n + \frac{c n}{r} - h\right]$

Cancelling the two terms, h and $-h$, in the brackets, and neglecting the term ..... $\frac{n c}{r}$, which contains two of the small quantities, c and n, and is added to the other terms which contain only one, there is obtained

$$B = \frac{p R S_o}{s}\left[-\frac{c}{s} - \frac{n}{s}\frac{h}{r}\right] = \frac{p R S_o}{r}\left[-\frac{r}{R} - \frac{h}{R}\frac{h}{r}\right];$$

that is, $$B = - p S_o \left[\frac{r^2 + h^2}{r^2}\right] = - \frac{R^2}{r^2} S_o p; \quad \ldots\ldots\ldots (82a)$$

which is equal to A, but of contrary sign.

A comparison of Equations 82 and 82a shows that the former can be obtained from the latter by simply adding the term, $p\ h$, which is nothing more or less than the horizontal (compressive) hoop stress which would occur in this ring if it were the crown ring of an open dome (see Equation 8, page 266), in which situation the hoop stress in the ring would be due solely to its own weight. It might now be contended, with some plausibility, that the term to be added to the expression for the hoop stress, B, just found on the supposition that the ring in question has no weight, in order to establish a value for B, holding good in case the weight of the ring is considered, should be this very term, $+ p\ h$, which is known to be the complete value of the hoop stress in a situation where the ring's own weight is the sole cause of hoop stress. Perhaps some such idea as this was in the author's mind when the treatment of Cases II and III was written; but the writer must confess himself unable to apprehend what the author means by the stress called B in Case II, *viz.*, "the horizontal ring stress, B, in the bed-joint of the base" of any dome. Since any horizontal ring of voussoirs acts as a base for the portion of the dome situated above it, it would seem as if the phrase "horizontal ring stress" must mean the hoop stress in any horizontal ring, *viz.*, $B = - \frac{R^2}{r^2} a p + p h$; but the author, evidently, considers that in Case II he has not reached the proper stage for announcing this value for B. As before remarked by the writer, the representation of some definite body, in the way of a voussoir or of a collection of voussoirs, some of the forces acting on which involve the B of the author's Case II, would seem necessary, if his meaning is to be made plain.

The obscurity in the treatment of the force, B, in Case II, is probably connected with Proposition *V*, on page 293. This proposition the writer holds to be a fallacy, which, while harmless (as regards final results) in the author's treatment of the spherical dome, gives rise to an erroneous expression for B in domes of other forms.

For example, in the case of the conical dome, results for which are stated at the foot of page 299, the author claims

$$A = + \frac{S^2}{2a} p \quad \ldots\ldots\ldots (70)$$

and $$B = - \frac{S^2}{2a} p + \frac{r^2}{a} p \quad \ldots\ldots\ldots (71)$$

(In Equation 71 the writer has put p for p_o and p_a, as is done by the author on page 300). But, in applying to the conical dome the same treatment as that just given in this discussion for the spherical dome,

Mr. Church. the writer finds that the results should be

$$A = + \frac{S^2}{2\,a}\,p \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (70a)$$

and

$$B = \frac{r^2}{a}\,p \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (71a)$$

from which it would appear that in all conical domes, whatever the angle of inclination, the circumferential hoop stress, B, is always compressive.

Of course, if Equation 71a, instead of Equation 71, gives the true value of B, then the author's conclusions, occupying the whole of page 300, are incorrect. According to one of these conclusions, the horizontal courses of a conical dome the sides of which are steeper than 45° with the horizontal are subjected to horizontal tension; and, according to Equation 71, as the angle increased the greater would be the tension. But, at 90°, the cone becomes a cylinder, of infinite height, in which case, of course, there is no hoop stress of either kind in the horizontal courses; but, from Equation 71, this hoop stress, for an angle of 90°, would be tensile, and infinite in value, whereas, from Equation 71a, B = zero, which is known to be true. That Equation 71 is erroneous, would therefore seem to be manifest.

As a result of the author's general expression for B, he reaches the result that in a truncated conical dome there exists a joint of rupture; but, in the opinion of the writer, this conclusion is altogether erroneous, for the reasons just given.

The detail of the writer's work in deriving Equations 70a and 71a is as follows:

In Fig. 41 is shown a small block or voussoir, $M\,N$ (of a full conical dome, homogeneous and of uniform thickness), being part of a horizontal ring-course the radius of the upper edge of which is r (of the lower edge, r') and bounded on the sides by two vertical meridional planes, $N\,C\,V$ and $M\,C\,V$, making with each other a small angle, θ. The elements of the cone make an angle, α, with the horizontal. A being the meridional thrust per unit length of "parallel," and B the circumferential, or hoop stress, per unit length of meridian, the four thrusts acting on the small block are $A\,r\,\theta$, $A'\,r'\,\theta$, and two side thrusts (each $= B\,s$). S is the slant height, $M\,V$, and a the altitude, of the part of the dome above M.

First, to find A, consider the part of the dome above M. Its weight is $2\,\pi\,r\,\frac{S}{2}\,p$ (where p is the weight per unit area of the convex surface of the cone), and must be equal to the sum of the vertical components of all the thrusts in the curved edge of its base, the length of which is $2\,\pi\,r$. Therefore,

$$2\,\pi\,r\,\frac{S}{2}\,p = \left(A\;2\;\pi\;r\right)\text{sin. }\alpha;\ \text{or, } A = \frac{S\,p}{2\text{ sin. }\alpha};$$

whence,

$$A = \left[\frac{p\;S}{2}\right] \div \frac{a}{S} = \frac{S^2}{2\,a}\,p \ldots\ldots\ldots\ldots\ldots (70a)$$

In view of subsequent work, a more convenient form for A (more convenient, although "cloaked in the abominable trigonometrical functions") is obtained thus: Mr. Church.

$$\frac{S^2}{2a}p \text{ can be written } \frac{p}{2}\,\frac{S}{a}\,\frac{S}{r}\,r;$$

that is,
$$A = \frac{p\,r}{2 \text{ sin. } \alpha \text{ cos. } \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (83)$$

and therefore, also,
$$A' = \frac{p\,r'}{2 \text{ sin. } \alpha \text{ cos. } \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (84)$$

(This shows that A is directly proportional to the radius, r.)

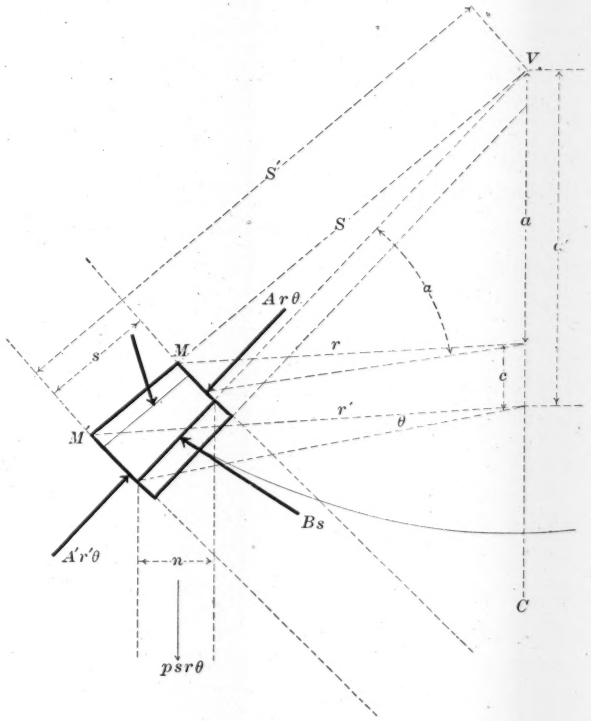


FIG. 41

Returning to Fig. 41 (in which the weight of the block, $M\ N$, is shown as $p\ s\ r\ \theta$, though not needed for subsequent work, as it has no horizontal component), it is noted that for equilibrium the system of

Mr. Church. forces obtained by projecting the actual forces on a horizontal plane will itself be a system in equilibrium. Fig. 40 will serve to represent the result of this projection (except that α' is equal to α in the present case). In Fig. 40, summing the components parallel to the radius bisecting the small angle, θ,

$$2\,B\,s\,\frac{\theta}{2} = A'\,r'\,\theta\,\text{cos.}\;\alpha - A\,r\,\theta\,\text{cos.}\;\alpha.$$

That is,
$$B\,s = \text{cos.}\;\alpha\,\left(A'\,r' - A\,r\right);$$

whence, from Equations 83 and 84,

$$B\,s = \frac{p\,\text{cos.}\;\alpha}{2\,\text{sin.}\;\alpha\,\text{cos.}\;\alpha}\left(r'^2 - r^2\right).$$

Now, $r' = r + n$;
therefore,

$$B\,s = \frac{p}{2\,\text{sin.}\;\alpha}\left(r^2 + 2\,r\,n + n^2 - r^2\right) = \frac{p\,n}{2\,\text{sin.}\;\alpha}\left(2\,r + n\right).$$

But, as the small distance, n, grows indefinitely small, it vanishes when added to $2\,r$, so that

$$B = \frac{n}{s}\,\frac{p\;r}{\text{sin.}\;\alpha} = (\text{cos.}\;\alpha)\,\frac{p\;r}{\text{sin.}\;\alpha} = p\,r\,\text{cotan.}\;\alpha = p\,r\,\frac{r}{a}.$$

Hence, finally,
$$B = \frac{r^2}{a}\,p \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (71a)$$

It is easily shown that in case the conical dome is truncated, whether with or without a lantern, the value of B given by Equation 71a remains unchanged; *viz.*, B still $= \frac{r^2}{a}\,p$.

The value of A, however, would be different from that given by Equation 70a, if the dome were truncated or had a lantern.

It may be of interest to state that the results, for the conical dome, derived by the writer, are in exact agreement with those given by Rankine in his "Applied Mechanics." Rankine, however, does not present details of derivation, but merely outlines a general method for all domes, making use of the principles of the differential calculus.

* In that part of the author's discussion where he deals with a truncated conical dome, page 323, he has evidently confused the two symbols, h and a (see Fig. 32). The distance, $a = C\,V$, is measured to the vertex, V, (real or imaginary) of the cone, and becomes infinity (while r_o becomes equal to r), when the sides of the conical frustum become vertical; in which case the writer's Equation 71a gives $B =$ zero (as it should), and not the author's Equation 85.

The author's equation for ring stress (Equation 74, on page 303), however, when the frustum becomes a vertical cylinder, in which case $r_o = r$, and $S = h$, gives the result:

$$B = -\,p\,h \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (86)$$

which is evidently not zero, but a tensile ring stress. In other words,

* Added by Professor Church after he had read the author's discussion on page 343.

if a dry brick wall is built in the form of a vertical cylinder, of altitude $= h$, with its own weight alone to support, it would be necessary, according to the author's formula, to provide hoops around the lower courses, to prevent bursting. Such an outcome is certainly very difficult to accept. Mr. Church.

H. H. WADSWORTH, M. AM. SOC. C. E. (by letter).—In attempting to follow the reasoning and develop the author's formulas in that part of the paper relating to the spherical dome with a homogeneous surface, the writer, being unable to check some of the expressions there given, or even accept some of the hypotheses upon which they are based, has worked out the problem in a somewhat different manner, and presents the results in the hope that they will, at least, bring out further discussion. Mr. Wadsworth.

The author's Equations 1 to 6, relating to the surface properties of a sphere, are correct and fundamental, though the reason for the development of Equation 6 is not apparent, as it merely expresses the condition previously assumed, *i. e.*, the weight of each square unit of the surface $= p$.

Let Fig. 42 represent a vertical section through the axis of a hemisphere, cut in two by a plane parallel with the equator at a height, h, above it. Then the weight of the portion of the hemisphere above the cutting plane is

$$2 \pi R a p,$$

and of the portion below that plane it is $2 \pi R h p$.

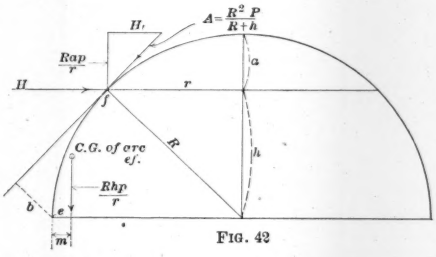


FIG. 42

Consider, now only, the spherical sector included between meridians at a distance apart of unity at their intersection with the cutting plane, and the weights of the respective portions will be:

$$\frac{2 \pi R a p}{2 \pi r} = \frac{R a p}{r}, \text{ and } \frac{2 \pi R h p}{2 \pi r} = \frac{R h p}{r}.$$

Before considering the sector of the whole hemisphere, take the upper portion only. The forces acting at the base of this are: The reaction of the support = the weight $= \frac{R a p}{r}$; the meridional tangential thrust, A; and a horizontal force, H.

From similar triangles, $A : \frac{R a p}{r} = R : r$;

or, $$A = \frac{R^2 a p}{r^2} = \frac{R^2 (R - h) p}{R^2 - h^2} = \frac{R^2 p}{R + h},$$

the value of A, as determined by the author.

Mr. Wadsworth.

Similarly, $$H_1 : \frac{R\,a\,p}{r} = h : r;$$

or, $$H_1\,r = \frac{R\,a\,p\,h}{r} = \frac{R\,(R-h)\,h\,p}{\sqrt{R^2-h^2}}.$$

But $H_1\,r = B_1$ = the ring stress at this point (see the author's Equation 1); therefore, $$B_1 = \frac{R\,(R-h)\,h\,p}{\sqrt{R^2-h^2}}.$$

In this equation, when $h = R$, or when $h = 0$, $B_1 = 0$. Equating the first derivative of this expression to zero gives the value of h, which makes B_1, maximum, $h_1 = 0.618\ R$, and, for this value of h, $B_1 = 0.3\ R^2\,p$.

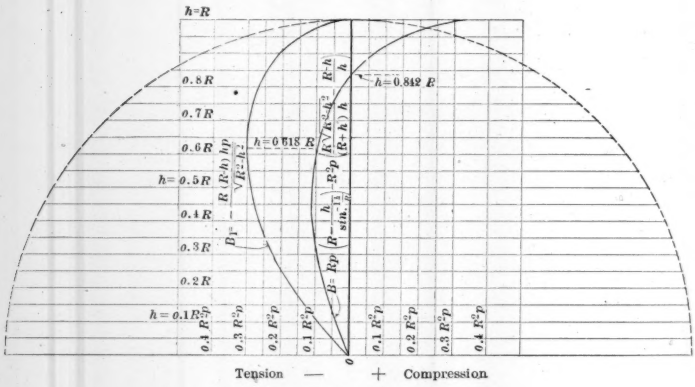


FIG. 43

In Fig. 43 the abscissas to the curve marked $B_1 = -\frac{R\,(R-h)\,h\,p}{\sqrt{R^2-h^2}}$ are the ring stresses (tension) at bases of domes in terms of $R^2\,p$, for values of h (the height of the base above the equator) as ordinates.

Referring again to Fig. 42, and remembering that, since the ring stress at the equator is zero, as just determined, there will be no tendency for the dome to spread at this point, there will be, acting on the lower portion of the sector of the dome, the tangential meridional thrust, $A = \frac{R^2\,p}{R+h}$, the weight of the lower portion, itself $= \frac{R\,h\,p}{r}$, acting at the center of gravity of the arc, $e\,f$, and the force, H, acting in the cutting plane.

To maintain equilibrium, the algebraic sum of the moments of these forces must be zero.

Mr. Wadsworth.

Taking as the point of moments the equatorial base of the sector:

$$H\,h - \frac{R\,h\,p\,m}{r} + \frac{R^2\,p\,b}{R+h} = 0,$$

or,

$$H = \frac{R\,p\,m}{r} - \frac{R^2\,p\,b}{(R+h)\,h}.$$

Calling the ring stress at this point, B, then

$$B = H\,r = R\,p\,m - \frac{R^2\,p\,b\,r}{(R+h)\,h}.$$

From Fig. 42 it is readily seen, or easily demonstrated, that $b = R - \sqrt{R^2 - h^2}$.

m may be determined by an application of the theorem that the area of a surface of revolution is equal to the length of the generating curve multiplied by the distance described by the center of gravity of the curve.

In this case, the length of the arc $= R \sin.^{-1} \frac{h}{R}$, and the distance traveled by the center of gravity is $2\,\pi\,(R - m)$. The area of the zone has been found to be $2\,\pi\,R\,h$;

therefore,

$$2\,\pi\,(R-m)\,R \sin.^{-1}\frac{h}{R} = 2\,\pi\,R\,h;$$

$$R - m = \frac{h}{\sin.^{-1}\frac{h}{R}}, \text{ and } m = R - \frac{h}{\sin.^{-1}\frac{h}{R}}.$$

Then, in the equation,

$$B = R\,p\,m - \frac{R^2\,p\,b\,r}{(R+h)\,h}$$

substitute for m, its value, $R - \frac{h}{\sin.^{-1}\frac{h}{R}}$;

substitute for b, its value, $R - \sqrt{R^2 - h^2}$;

and substitute for r, its value, $\sqrt{R^2 - h^2}$;

and the result is

$$B = R\,p\left(R - \frac{h}{\sin.^{-1}\frac{h}{R}}\right) - R^2\,p\left[\frac{R\sqrt{R^2-h^2}}{(R+h)\,h} - \frac{R-h}{h}\right].$$

The curve represented by this formula is also shown in Fig. 43, the ring stress at any parallel, a height, h, above the equator, is given by the abscissa to the curve at that point.

It will be noticed that this curve crosses the zero line of stress at a height above the equator of $h = 0.842\ R$, instead of at a height of $0.618\ R$, as determined by the author; and it will be noticed, also, that this latter height, as already pointed out, is the position of the base of a dome, less than hemispherical, which gives the maximum base-ring stress.

The curve of ring stresses in hemispherical domes shows the maximum tension to be at $h = 0.5\ R$, approximately, and that the tension at this point is $0.11\ R^2\,p$.

Mr. Wadsworth.

For a practical application to a masonry dome, let t = the thickness of the masonry, in feet, also the unit of surface measurement.

Let w = the weight of the masonry per cubic foot;

then $$p = w\,t.$$

Let R_1 = the radius of the dome, in feet (R being the radius expressed in the units of which p was the weight of a square unit),

then $$R = \frac{R_1}{t}.$$

Then, in the expression, $0.11\ R^2\ p$, substitute for R and p the values just found, and the result is

$$\frac{0.11\ R^2_1\ w\ t^3}{t^2} = 0.11\ R^2_1\ w\ t.$$

If the tensile strength of the stone be K pounds per square inch, or 144 K pounds per square foot, the strength of a ring having a width and thickness of t will be $144\ K\ t^2$;

then $$0.11\ R^2_1\ w\ t = 144\ K\ t^2.$$

Solving for K, gives $$K = \frac{0.11\ R^2_1\ w}{144\ t}.$$

According to Table No. 3, the dimensions of the dome of the Santa Sophia Cathedral at Constantinople are $R_1 = \frac{115}{2} = 57.5$ ft., and $t = 2\frac{1}{2}$ ft. (this is the thickness at $h = \frac{R}{2}$), assuming that the thickness varies uniformly from 2 ft. at the top to $2\frac{3}{4}$ ft. at the base;

then $$K = \frac{0.11 \times (57.5)^2 \times 160}{144 \times 2\frac{1}{2}} = 162 \text{ lbs. per square inch,}$$ tension in the masonry ring. (The weight of the stone per cubic foot is assumed at 160 lbs. $= w$.)

Of course, the fact that this dome is built with a varying thickness from base to top, and that it is buttressed, renders the formulas herein derived not exactly applicable; but it would seem that the builder did not depart widely from good practice, as far as the strength of the stone itself is concerned.

Further, the fact that, at the base, the ring stress is zero, makes the openings, of which the author states there are forty, unobjectionable.

Mr. Schmitt.

E. SCHMITT, ASSOC. M. AM. SOC. C. E. (by letter).—Attention is called to the fact that Rankine, Schwedler and Dr. Robison are mentioned in the paper as those who solved the problem of the stresses in a spherical dome "a long time ago."

As to the use of trigonometrical functions in practical working formulas, the writer will give two examples:

1.—The ring stress in a hemispherical dome, at any altitude, is:

By Schwedler: $$B = p\ R\left(\frac{1 - \cos.\ \alpha - \cos.^2\ \alpha}{1 + \cos.\ \alpha}\right)$$

Mr. Schmitt.

In the paper: $$B = -\frac{R^2}{R+h}p + p\,h \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

Wherein α = the angle from the vertex, and h = the elevation of the joint in question above the equator.

2.—The thrust, by a single load, on a two-hinged circular arch, is given in the following forms:*

By Winkler:

$$H = W \times \frac{\sin.^2\alpha - \sin.^2\beta + 2\cos.\alpha(\cos.\beta - \cos.\alpha) - 2\cos.\alpha(\alpha\sin.\alpha - \beta\sin.\beta)}{2[\alpha - 3\sin.\alpha\cos.\alpha + 2\alpha\cos.^2\alpha]};$$

By Engesser: $$H = W\frac{a\,R(l-a) - 2\,h(c-c_1)\,a - h\,l\,c_1 + 2\,R\,h\,d}{2\,R^2\,c + 4\,h^2\,c - 3\,h\,R\,l}.$$

Professor Church has called attention to the following paragraph in relation to Case II, on page 266.

"If B, the resisting or balancing force, as it were, is assumed to be reckoned per unit length of circumference, and for one unit of length along the meridian," etc.

When writing this "conclusion" the writer had in mind the co-related behavior of the shearing stresses on the four faces of an element, considered free, and which are all, and are always, of the same intensity per unit of area.

This theorem is demonstrated in all handbooks on applied mechanics, and the writer sees no reason why, in this instance, it should fail to work.

All who are interested are referred to Professor Church's "Mechanics of Engineering,"† where, under "Shearing Stresses," will be found the theorem as stated above.

Now, in this case, the "element" is a spherical square, situated at the base of the dome, and of which each side measures one unit of length.

If, now, the shearing stresses, along the meridional sides of this element, are caused by the thrust, A (acting per unit of length of the circumference), is it true, or untrue, that the ring stress, B, per unit of length along the meridian, is of the same intensity as the stress, A, the unit of area being common to both?

If this reasoning is "loose and elusive," Professor Church may, possibly, remove some of the obscurity as to the meaning and

* Wherein, in Winkler's formula:
W = the single load;
α = the angular measure of half of the arch;
β = the angular measure of the load point from the vertex.
In Engesser's formula (origin in one of the hinges);
l = span;
h = radius, minus rise of arch;
R = radius;
a = abscissa of the load point;
d = ordinate of the load point;
c = half the length of the arc;
c_1 = the length of the arc, from the origin to the load point.

† Edition of 1888, paragraph 209, page 228.

Mr. Schmitt. applicability of this theorem, as the qualifications as to clearness would attach thereto, if it cannot be made use of, in this case, also.

The following theorem is submitted in proof of the statement that, at the base of any dome, the tensile ring stresses, and also the shearing stresses, are all of the same intensity, and equal to the known meridional compressive thrust, A.

Theorem.—When, upon two opposite sides of a rectangular element (whereof each side measures one unit of length), there acts an "applied" compressive force, of a magnitude $= A$, and "induces" upon the other two sides a tensile stress, B, then this induced stress is of the same magnitude as the applied force. The shearing stresses along the four sides of this element must also be of the same intensity.

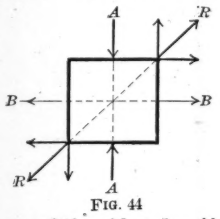


FIG. 44

In Fig. 44, considering equilibrium in a diagonal direction, the resultants of the compressive and tensile stresses (of two adjacent sides), must pass through opposite corners of the element, and have an inclination of 45° to any of the sides, for, if this should not take place, the shearing stress components of the resultants would not be of equal magnitude, which, however, for equilibrium, they must be. It also follows that all the stresses have the same intensity.

As to the stresses in spherical and conical domes, given in the second part of the paper, and there derived from the general case of the pyramoidal dome, it will be necessary to prove that the formulas for this general case are defective, before making exceptions as to the results obtained from them for conical domes.

Professor Church finds, for the compressive ring stress, "at any altitude," in a conical dome:

$$B = \frac{r^2}{a} p \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (71a)$$

This is wrong. This expression represents simply the compressive ring stress at the top of a truncated dome. The writer would suggest that, in this case, the tensile ring stress at the base of a given cone be found first, and then the compressive ring stress at the top of a truncated cone, both in reference to a common radius, and then add the results.

Professor Church finds the ring stress in all conical domes, whether complete, truncated, with or without a lantern load, and at any angle of inclination, to be always a circumferential compressive stress, and equal to

$$B = + \frac{r^2}{a} p \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (71a)$$

Mr. Schmitt.

Assuming a self-contained conical dome, standing upon an open-jointed, circular, stone base (see Fig. 45), the question arises: What must be the magnitude of the tensional resistance in the bottom course of this cone? There is no doubt that this stress is a tensile one; otherwise, the dome would spread, and would not be self-contained. The next question is: What induces this tensional stress? Is it the inclined reaction, A, or is it due to some other applied force, and what is that force? If A, as the writer claims, is this force, and induces tension, then the ring stress, B, will increase as A increases, say, for instance, through the action of a lantern load.

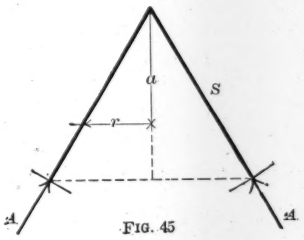


FIG. 45

Continuing, Professor Church writes:

"But, at 90°, the cone becomes a cylinder, of infinite height, in which case, of course, there is no hoop stress of either kind in the horizontal course; but, from Equation 71, this hoop stress, for an angle of 90°, would be tensile, and infinite in value."

This can be admitted as the most likely thing to occur, since the value of A also becomes infinite. Not much of the cone would remain in that instance, at least not much of the bottom course. But, assuming, also, with Professor Church, that a truncated cone, just one unit of measure in height, and with an inclination of 90°, is to be dealt with, then, evidently,

$$B = + r^2 p \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (85)$$

That is, this cylinder is subjected to a compressive ring stress, increasing with the square of its radius. This result, therefore, would seem to be manifestly erroneous, as far as Professor Church is concerned, because, as he writes, there are no ring stresses in cylinders standing on end. Yet this contradictory result is obtained by following Professor Church's reasoning, and by the use of the formula established by him.

Professor Church seems to be given somewhat to hair-splitting, when he objects to having apparently combined the action of stresses not located in the same parallel of altitude, but instead just above and just below the same, respectively, and as was done in the paper in the case of the ring stresses at any altitude in a spherical dome, under Case III. With a little imagination it is to be perceived that when it is wished to discover the combined action of these stresses for a mere point, then the courses in which these stresses take place, and are to be combined, coincide, that is, overlap one another.

Professor Church cites Rankine's "Applied Mechanics" in corroboration of the correctness of the results obtained by him for conical domes.

Mr. Schmitt. Referring to Equation 15 for the ring stress at any altitude in a spherical dome, namely:

$$B = -\frac{R^2}{R+h}p + p\,h \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

and applying the same to the parallel of altitude, $h = 0.618\ R$, that is, at the joint of rupture, then the ring stress, B, is found to be $= 0$. However, Rankine * writes:

"The angle of rupture, for which $p_z = (B) = 0$, is

$$i_o \text{ arc} \cdot \cos. \frac{\sqrt{5}-1}{2} = 51^\circ\ 49' ; \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

and from this angle we obtain, for the horizontal thrust of the dome, per unit of periphery at the joint of rupture,

$$p_y = 0.382\ p\ r\ ;$$

and for the tension on a hoop to resist that thrust,

$$P_y = 0.3\ p\ r^2. \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)"$$

The writer believes that Professor Rankine is here in error.

As mentioned, there is no ring stress at the (misnamed) joint of rupture; consequently no hoop is needed there. The compressive effect of the (overhanging) top course of the lower part, offsets, by an equal amount, the tensile, spreading, effect produced in the bottom course of the upper part.

The facts in the case are these: If the dome extended from the crown no further than to the joint of rupture, and were to form a self-contained shell, then a hoop would be needed around the base, to accomplish this end. The hoop would have to resist a tensile stress of $B = 0.618\ R\,p$, and not, as Rankine has it, of $0.3\ R^2\,p$.

If, however, the dome is a complete half sphere, and, also, is assumed to be a self-contained shell, then the lower part (below the joint of rupture) must be designed so as to conform in strength to the requirements of Equation 15, or be reinforced by hoops.

If the tension zone (the lower part) of the dome is now constructed as stated, it will resist effectively all the meridional downward pressures brought to bear upon it from above, and this without the assistance of a hoop at the joint of rupture.

Having thus shown that Rankine has evidently made a statement, relative to the stresses in spherical domes, not borne out by the facts, it is possible, therefore, that his deductions for the ring stresses in conical domes, also, are erroneous.

To answer Mr. Wadsworth's discussion properly, it is necessary to have a clear conception of the sense and direction, and of the magnitude of the external forces, acting, in the one case, in the top course of the lower part, and, in the other case, at the base of the upper part.

The two cases are radically different, as regards the question of determining resultants and components.

* "A Manual of Applied Mechanics," p. 267.

Mr. Schmitt.

In Fig. 46, which represents the external forces acting at the top of an open dome, the forces, H_o and A, are the components of the initial force (weight) p.

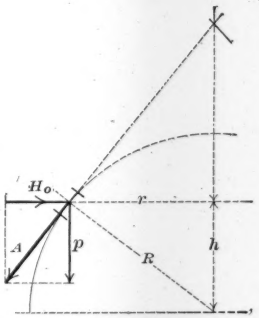


FIG. 46

Therefore, when these components, H and A, are considered in their action upon the top course, they at once represent the effect of p, that is, p drops out of the computation in this case.

Now, since the inclined meridional component force, A, is taken up, directly, by the next lower course, the component, H, is the only force to be considered as having an effect upon the top course, as in accordance with Case I.

In Fig. 47, which represents the external forces acting at the base of the upper part of a dome, H and W are the components, and A is the resultant of these two.

In this case, therefore, it is necessary to enter into the computations, either the two forces, H and W, together, or else only the inclined meridional upward reaction, A. Mr. Wadsworth neglected to analyze the external forces in this manner in both instances, and did not consider that, in an open dome, the top course forms an equilibrated ring, in compression; further, that the weight and the stresses in the lower courses have no effect whatever upon any of the upper courses, except in the case of two superimposed (*i. e.* overlapping) courses, as under Case III.

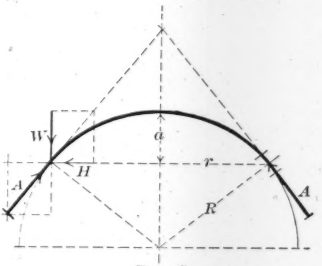


FIG. 47

On the other hand, all lower courses are affected by the upper ones, that is, by the inclined meridional thrust, A, due to the weight of these courses.

In the lower supporting courses, and below the joint of rupture, tensile ring stresses are created thereby, and, in the case of a hemispherical dome, a tensile stress at the base is produced equal to

$$-Rp \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

Mr. Wadsworth has treated the lower part of a spherical dome as if it were a part of a cylindrical vault. The stresses in a dome have nothing in common with the stresses in an arch. It is a mistake to assume the whole of the lower part of the dome to have an effect upon the bottom course of the upper part.

The writer considers all the formulas established for the stresses in domes in the paper to be correct.

Mr. Schmitt. * While, from a mere "mathematical point of view," the intersection of the side of a cone with its axis, at an inclination of 90°, is at infinity, and therefore a is of infinite height, and, according to Professor Church's Equation 71a, the ring stress, B = zero, such a happening, of an infinite height of the resulting cylinder, is not to be looked for, from a "practical point of view."

To compare the (ring) stresses in cylinders with those taking place in cones, it is but fair to assume that they have the same height.

In this sense the writer has to make a correction in the paragraph in the center of page 323, in that, instead of a "truncated cone," simply "a right cone," "one unit measure in height," is to be understood.

The example chosen by Professor Church, of a "dry brick wall * * * in the form of a vertical cylinder," requiring, "according to the author's formula," hoops around the lower courses, "to prevent bursting," and that such an outcome is very difficult to accept, is to be answered as follows:

If this circular wall is limited to "practical" heights, the ring stress, B, is so small that no hoops will be needed to counteract it; but when it comes to the question of carrying such a circular tower of brick up to its "critical (crushing) height" (not to mention infinity), then the writer is confident that hoops will be needed indeed, to force the loosened particles of the brick wall to maintain the cylindrical form of the original mass.

The required hoops would have to resist a tension of:

$$B = -p\,h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (86)$$

It is well known that "a cube" of stone, when tested to ultimate strength, attains, in the direction of the applied force, the shape of two truncated pyramids, meeting in their smaller ends, the other four, side-surface, pyramids having been forced out, gradually and in layers, in a direction normal to that of the applied force.

It is evident that this successive forcing out of the layers of stone is possible only after the tensional cohesion and resistance between the parts in place and those forced out has been, step by step, overcome.

In ordinary cases, therefore, of the foregoing example, such a tensional ring stress, B, is not "visible," although Professor Church seems to think that, to be perceivable, this ring stress ought to manifest itself in this manner. Attention is called to his remark "to prevent bursting."

The contrary, that such a tensional ring stress or tendency "does not exist," is by no means proven thereby.

In the example just discussed, the theorem stated near the top of page 322, is applicable, and will also lead to the expression:

$$B = -A = -p\,h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (86)$$

* Added by the author after reading the last paragraphs of Professor Church's discussion.

Mr. Schmitt.

By proper interpretation of the cited behavior of stone cubes, tested to their ultimate strength, it would seem permissible to say that the law of "hydrostatic pressure" is applicable to solid bodies also. Equation 86 is nothing else than the algebraic expression of this law.

Rankine's attitude in this matter, when he treats of the depths to be given to earth foundations, is well known.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 974.

METHOD USED BY THE RAILROAD COMMISSION, OF TEXAS, UNDER THE STOCK AND BOND LAW, IN VALUING RAILROAD PROPERTIES.*

By R. A. Thompson, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. E. L. Corthell, F. Lavis, W. H. Coverdale, W. D. Taylor and R. A. Thompson.

The issuance of all kinds of railroad securities in Texas is regulated by the Railroad Stock and Bond Law, and the execution of its provisions is under the jurisdiction of the Railroad Commission. In this matter, Texas is the pioneer of the States of the Union, no other State within the knowledge of the writer, having legislated upon the subject to the extent of having prescribed a fixed basis of value beyond which the railroads cannot increase the indebtedness to be secured by mortgage or lien upon their property. The law of Texas, in addition to controlling and regulating absolutely the issuance and execution of all stocks and bonds of railroads, defines the method which must be used by the Railroad Commission in arriving at the proper value of the properties, which shall serve as the basis for such issuance.

* Presented at the meeting of February 3d, 1904.

The Texas Railroad Stock and Bond Law was enacted in 1893, to supplement and strengthen the general Railroad Commission Act, passed by the State Legislature in 1891, which placed in the hands of a Commission, among other things, the entire control over, and regulation of, all freight rates and tariffs. The necessity for a law, limiting, on some reasonable basis, the capitalization of the railroads, and operating in connection with the control of freight charges, was obvious after some certain judgments which had been handed down shortly before by the United States Federal Courts. These decisions held, in effect, that any State tribunal affecting to control and regulate the charges on railroads for freight transported within the State, must permit the railroads to earn, in addition to the current expenses of maintenance and operation, the interest on their outstanding bonds and a fair dividend on their stock. It was considered that if the Commission elected to regulate freight charges was to be effective, it must have supervision and control over the issuance of stock and bonded securities, otherwise, by "watering" and other popular methods in vogue, the railroads could make their indebtedness as high as they chose, thereby preventing, in accordance with the decisions of the Courts, any reduction in rates that the Commission might consider reasonable.

Another reason for the enactment of the Stock and Bond Law was that the railroad securities of the State might have a real and substantial value, and that innocent purchasers might be protected. This feature of the law has been stated admirably by the Hon. John H. Reagan, ex-Chairman of the Railroad Commission of Texas, in the Second Annual Report of the Commission (1893). He says:

"This law will prevent the unlawful and fraudulent issue of stocks and bonds of railroad companies in the future in this State, and protect the people from the exactions necessary to satisfy them. It will protect the purchasers of such securities from being imposed on by them, and it will secure, to the companies who may hereafter build railroads, a real value to their property and securities, instead of a merely speculative and fluctuating value."

The Stock and Bond Law.

The Railroad Stock and Bond Law, as published under Title XCIV, Chapter 14, Revised Statutes of Texas, 1895, declares:

"Article 4584 a.—Among other things, the power and authority of issuing or executing bonds, or other evidences of debt, and all kinds of

stock and shares thereof, and the execution of all liens and mortgages, by railroad corporations in this State, are special privileges and franchises, the right of supervision, regulation, restriction, and control of which has always been, is now, and shall continue to be vested in the State Government, to be exercised according to the provisions of this and other laws.

"Article 4584 b.—Hereafter no bonds or other indebtedness shall be increased or issued or executed by any authority whatsoever, and secured by lien or mortgage on any railroad or part of railroad, or the franchises or property appurtenant or belonging thereto, over or above the reasonable value of said railroad property; provided that in case of emergency, on conclusive proof shown by the company to the Railroad Commission, that public interests or the preservation of the property demand it, the said Commission may permit said bonds, together with the stock in the aggregate, to be executed to an amount not more than fifty per cent. over the value of the said property."

It further provides, with regard to the railroads constructed and in operation at the time of the passage of the law:

"Article 4584 c.—It shall be the duty of the Railroad Commission to ascertain, and in writing report to the Secretary of State, the value of each railroad in this State including all its franchises, appurtenances and property. After it shall have prepared said report of value, the Commission shall give the company interested ten days' notice in writing, by registered letter to the president, treasurer or receiver of said railroad, to the effect that said report is ready to be made, and that if it have any objections thereto it must file them in writing, within forty days after said service, or the same will be so deposited with the Secretary of State as correct. Should the company or its duly authorized representative file with said Commission any objections to said report of value, the Commission shall duly investigate and pass on the same. On investigation, if the Commission conclude that its report of value is too low or too high, then it shall make the necessary correction before filing it. Should no objections be filed within the time permitted, or being filed and on examination found without merit, the Commission shall forthwith file its said report in the office of the Secretary of State, where it shall remain as a public record, as a limitation for the issuance of indebtedness under the limitations prescribed in Article 4584 b. To promote public interests and protect private rights, the Commission, after due notice under the rule herein prescribed, may correct its report of value of any railroad at any time it may deem proper."

With respect to railroads constructed after the passage of the law, or existing at that time, but against which stocks and bonds were not outstanding in excess of their value, the following section defines the method of procedure to be had in making the application for

issuance of stocks and bonds, and defines the basis on which the Railroad Commission must make its valuations:

"Art. 4584 f.—Should any company or corporation, authorized to construct, own or operate a railroad in this State, desire to issue bonds or other indebtedness, to be secured by lien or other mortgage on its franchise and property, in advance of the completion of the said railroad, it shall make application to and first procure the consent of the Railroad Commission thereto. In said application it shall exhibit to the Commission its contract with the construction company, if it have any; the profile of its completed road or part of road, the evidence of its right of way, depot grounds, terminal facilities; the extent and value of work done or in process of completion; the amount of property received; the amount of stock subscribed and the amount paid in; and all other necessary facts showing the value of the franchises and property proposed as security for said contemplated debts. If, on investigation, the Commission is satisfied that the company is acting in good faith, and that its contract with the construction company is reasonable and fair to the public, then it shall authorize the execution of said indebtedness and lien to the extent necessary for the demands of the work, at no time to be more than fifty per cent. over the value of the whole property and franchises."

After authority has been obtained from the Commission to issue stocks and bonds upon a railroad, and the same or any portion of it has been completed, Articles 4584 h and 4584 i of the Statute, which it is not necessary to quote here, prescribe the method of procedure to have the stocks and bonds registered in the office of the Secretary of State. It is only after such registration that the securities become a valid obligation against the property which is mortgaged. Stocks and bonds are approved and authorized to be registered by the Commission only upon and to the actual value of the completed railroad, as the same is determined by inspection.

Valuation of Railroads Existing at the Time of the Passage of the Law.

In accordance with the requirements of Article 4584 c, of the Stock and Bond Law, it became the duty of the Railroad Commission to value the railroads of the State then existing, and to report their values to the Secretary of State. The basis for valuation is practically outlined in Article 4584 c, and was interpreted by the Commission to mean, the estimated cost of duplicating or reproducing the properties at the time of valuation, allowing current market prices for labor,

materials and equipment, and a fair value for all right of way, depot grounds and other real estate used for railroad purposes.

The engineers of the Commission employed to make these valuations had for their guidance in making estimates of quantities of grading, etc., only the profiles of the railroads, which were required to be filed with the State under a previous statute. The railroads at that time were not favorably disposed toward the Commission and the Law, and it was difficult to secure information as to their original cost, even where the same was contained in their official records. In many instances, the railroads had passed through several managements and receiverships, and the original construction notes and records had been misplaced or were entirely lost. In several cases all records had been destroyed by fire.

The Railroad Commission appointed engineers to assist in making the valuations, and they, with the profiles referred to, in hand, made a detailed inspection of the railroads on the ground. The quantities of excavation and embankment, where the actual quantities could not be obtained, were estimated approximately from the profiles, using the center heights of the cross-sections. The classification of the materials in excavation was determined by inspection. Where the original plans and estimates of cost of the bridges, buildings and structures of all kinds could not be obtained from the records of the railroads, their value was estimated from measurements taken on the ground. The extent and acreage of the right of way, the depot and terminal grounds, were determined by actual measurement, or from maps furnished by the railroads, or from the city and county tax records.

After an examination of a railroad had been made by the engineers of the Commission, its valuation was prepared on estimate sheets. Upon sheets marked Estimate Sheet A, and headed "Estimate of the Value of the ——— Railroad, from Mile No. — to Mile No. —," were recorded the values of the right of way and depot grounds, roadbed, track, bridges, structures and way buildings for each mile, the value of ten miles being recorded on each sheet. (See Items on Sheet A.) On these sheets space was provided for the units and prices, and columns for carrying out the values for each mile and the totals.

The value of all rolling stock and equipment, and the value of such properties as were properly applicable and chargeable to the entire railroad, were recorded on a separate estimate sheet, only one sheet being used for a railroad.

Estimate Sheet A.

Estimate of the Value of the ..... Railroad, from Mile No. .... to Mile No. .....

Right of Way.

Depot Grounds. Exclusive of Round House and Machine Shop Sites and Terminal Facilities.

Graduation:
- Clearing and Grubbing.
- Original Excavation: Earth. Gravel. Loose rock. Solid rock.
- Tunnels.
- Original Embankment: Hauled. Earth. Gravel. Loose rock.
- Widening excavation.
- Widening embankment.
- Rip-rap.
- Retaining walls.

Bridges:
- Foundations: Excavation, Earth. Gravel. Loose rock. Solid rock. Wet.
- Coffer-dams, Pumping, Caissons, etc.
- Piling.
- Grillage.
- Concrete.
- Masonry: Abutments and Piers, First class. Second class. Third class.
- Cylinders (length and diameter).
- Culverts, First class. Second class. Third class. Dry.
- Trusses: Iron and Steel, Through trusses (length).
- Iron and Steel, Deck trusses (length).
- Combination trusses (length).
- Wooden trusses (length).
- Trestles: Trestle bridges (length).
- Piling for Trestles (length).
- Wooden box-drains (length).
- Earthenware drain pipe (length and diameter).
- Wooden cattle guards.
- Iron cattle guards.
- Fencing right of way.

Track:
- Ties.
- Steel rails.
- Iron rails.
- Joints, complete.
- Spikes.
- Laying, including expense of two trains.
- Hauling ties.
- Surfacing.
- Ballasting.

Telegraphs. Line complete.

Buildings:
- Exclusive of general offices, machine shops, round houses, etc.
- Passenger depots, including platforms.
- Freight depots, including platforms.
- Cotton platforms.
- Sundry warehouses.
- Train sheds.
- Section houses.
- Water stations.
- Stock pens.
- Coal chutes.
- Wharves.
- Turntables.
- Track scales.
- Road crossings and sidewalks.
- Signs, of all kinds.

On the sheet for the rolling stock, space is provided for the description, number, rate and value of all locomotives and cars.

The locomotives are classed as "Passenger," "Freight" or "Switch Locomotives," and there are columns headed as follows: Gauge, Maker, Size of Cylinders, Wheel Base, Length of Engine, Total Length of Engine and Tender, Weight on Drivers, Weight of Engine, and Total Weight of Engine and Tender.

The cars are classed as follows:

Passenger Cars.	Freight Cars.	Maintenance and Other Cars.
Passenger cars, first class.	Box cars, 34 ft. long and over.	Officers' cars.
" " second class.	" " 32 to 34 ft.	Office cars.
Sleeping cars.	" " less than 32 ft.	Pay cars.
Chair cars.	Stock cars, 34 ft. long and over.	Boarding cars.
Dining cars.	" " 32 to 34 ft.	Water cars.
Buffet cars.	" " less than 32 ft.	Wrecking cars.
Baggage cars.	Flat cars, 32 ft. and over.	Derrick cars.
Postal cars.	" " less than 32 ft.	Pile-driver cars.
Baggage and mail combined.	Gondolas.	Excavator cars.
Express cars	Coal cars, 60 000 lbs. capacity.	Dump cars.
Combination passenger and baggage cars.	" " ordinary.	Iron cars (track laying).
Emigrant cars.	Tank cars.	Push cars.
	Refrigerator cars.	Hand cars.
	Cabooses.	Velocipedes.

Other items are included in the following:

Maintenance of Way Dept.:
Excavators on cars.
Wheel scrapers.
Drag scrapers.
Plows.
Other road tools in use.
Other tools in storehouse.

Bridges and Buildings Dept.:
Pile-drivers on cars.
Pile-drivers, ordinary.
Derricks on cars.
Derricks, ordinary.
Barges and scows.
Other bridge tools in use.
Other bridge tools in storehouse.

Machine and Repair Shops:
Stationary engines and other power in shops.
Machine tools.
Other shop tools in use.
Other shop tools in storehouse.

Transfer tables.
Travelers.
Other hoisting appliances.

Franchises and Real Estate:
Other than right of way and depot grounds proper.

General offices.
Machine and repair shops.
Round houses.
Terminal grounds.
Other yard extensions.
Hospitals.

State charter.
County franchises.
Municipal franchises.

Miscellaneous:
Legal expenses during construction.
Engineering expenses during construction.
Superintendence during construction.
Damages contingent on accident.
Damages contingent on land.
Furniture and fixtures, general offices.
Furniture and fixtures, elsewhere.
Stationery.
Interest during construction.

Materials and Supplies:
Roadway Dept.
Bridges and Buildings Dept.
Machine and Locomotive Dept.

The values applied to the several unit items of construction, materials and equipment were the current market (contract) prices for the same, a liberal allowance being made for contingencies. The values applied to the right of way and other real estate owned by the railroads, but used strictly for railroad purposes, were in accordance with the current market value of the adjoining property. No deductions were made for any property which had been donated or which had been acquired by the railroads at less than its market value, but all was valued on the basis described. The value applied to the grading quantities was the outside market (contract) price prevailing. The values applied to lumber, rails, and the other materials of construction were the market quotations, with due allowance for freight charges to the point of construction. No deduction was made for depreciation in the value of materials on account of age, wear and tear, usage, etc., it being considered that the roadbed, track and all structures must be maintained in first-class and serviceable condition, and renewed from time to time, whenever necessary. On the other hand, no additional allowance was made for the value of seasoned roadbed, etc.

In accordance with the instructions of the Commission, no allowance was made for franchises of any kind except municipal franchises, and in that case only where the streets of a city or town were occupied by the tracks of a railroad in lieu of the purchase of adjoining property. The value then applied to such a franchise was a certain percentage of the estimated market value of abutting properties on each side of the street, less the value of the improvements thereon, usually about 33⅓ per cent.

An arbitrary allowance, usually from 5 to 6% of the total estimated value of the railroad, was made to cover all the items "legal and engineering expenses and superintendence." A similar allowance was also made for the item "interest during construction."

These valuations, after being summarized, were reported to the Commission, and, upon being adopted by it, were transmitted to the office of the Secretary of State for permanent record in his office, due notice, in accordance with the statute, having been given to the railroads.

Valuation of the New Railroads Upon Which the Issuance and Registration of Stocks and Bonds Is Applied For.

In accordance with the provisions of Article 4584 f, of the Statute, heretofore quoted, whenever any new corporation desires to construct a railroad, or whenever any existing company desires to make extensions of its system, authority can be had from the Railroad Commission to issue stocks and bonds in advance of the completion of the railroad, provided the rules and regulations of the Commission governing in such matters are complied with. These require that at least an actual survey of the proposed railroad shall have been made, and sufficient right of way secured, either by deed or contract, to show the good faith of the company. This authority to issue bonds, secured from the Commission in advance of the completion of the railroad, however, is of little value to the company except as a guaranty that when a certain railroad is completed in accordance with the plans and specifications filed with the application for issuance of stocks and bonds, the Commission will approve and direct their registration in the office of the Secretary of State, after which they become a valid obligation against the property. This authority, upon several occasions, has been used as the basis for construction and equipment contracts.

From the plans, specifications, contracts and other exhibits filed with an application for issuance of stocks and bonds, an estimate of the ultimate physical value of the railroad, when completed, is made by the Commission, upon the basis heretofore described, and the issuance of stocks and bonds authorized to the amount of the estimate. As before, due allowance is made for the value of all property which is to be donated to the railroad, which is to be used for strictly railroad purposes, and for municipal franchises which are to be actually occupied.

The estimate upon which the issuance of stocks and bonds is based is made up on Estimate Sheets A, B and C, which are prepared by the Commission. One sheet, of the form "Estimate Sheet A", is used for each mile of railroad, and, in the first column, under the heading, "Estimate Upon Which Authority to Issue Stock and Bonds Is Based," is placed the rate per unit item allowed, and the amount and value of each item of property and construction which is to enter

into the ultimate valuation of the railroad, except rolling stock and equipment, and such general items as are more properly chargeable to the entire line. These latter items are considered and the values applied are recorded on the form, "Estimate Sheet B and C," under the same heading as before, one sheet being used for the entire railroad. The total of these estimate sheets determines the amount of stocks and bonds that the Commission will authorize to be issued.

After the railroad, or any section of it, has been completed, *viz.*, the track laid and surfaced, application can then be made to the Railroad Commission for its approval and authority to register, of the stocks and bonds already authorized to be issued, an amount equivalent to the value of the completed railroad or section thereof. The Commission has its engineer make an inspection of the railroad reported to be completed, and, upon Estimate Sheets A, B and C, above mentioned, in the next column, under the heading, "Estimate for Registration of Stock and Bonds," is recorded the value, of the property acquired and construction completed, applicable. Upon approval of the engineer's report of value, the Commission orders the registration of the stocks and bonds.

As subsequent sections are completed, the Commission will authorize additional stocks and bonds to their value. Also, upon the completion at any time thereafter of any construction, or the acquirement of any property, contemplated in the original application upon which the issuance of the stocks and bonds was based, such as permanent bridges, ballasting, fencing, buildings, terminal grounds, etc., additional bonds will be authorized registered to their value as the same appears on the first estimate made by the Commission. The value of these latter items are entered in the last column of Estimate Sheets A, B and C.

In any case, however, whether or not the same was contemplated or was specified in the original application for issuance of stocks and bonds, since it is provided in Article 4584 b that the aggregate amount of stocks and bonds must not exceed the reasonable value of the property, any railroad can at any time apply to the Commission for authority to issue and register additional stocks and bonds to the amount of the value of any permanent improvements, betterments and equipments which have been added to the property or acquired by the company.

ESTIMATE SHEET; "A," ROADBED, STRUCTURES, ETC.

RAILROAD COMMISSION OF TEXAS.

*Rail*_____, *Mile No.*_____ *From*_____

ITEM.	Estimate Upon Which Authority to Issue Stock and Bonds is Based. Made.....190..				Estimate for Registration of Stock and Bonds. Made.....190..			Estimate for Registration of Stock and Bonds. Made.....190..		
	Rate.	Unit.	Amount.	Value.	Rate.	Amount.	Value.	Rate.	Amount.	Value.
Right of Way		Acre.								
Depot Grounds		"								
Clearing and Grubbing		"								
Earth Excavation		cu. yd.								
Gravel Excavation		"								
Loose Rock Excavation		"								
Solid Rock Excavation		"								
Earth Embankment		"								
Hauled Embankment		"								
Rip-Rap		"								
Foundation Excavation		"								
Foundation Piling		lin. ft.								
Grillage		B. M.								
Masonry Piers		cu. yd.								
Cylinder Piers		lin. ft.								
Masonry Culverts		cu. yd.								
Steel Trusses		lb.								
Plate Girders		"								
Trestle Timber		B. M.								
Trestle Piling		lin. ft.								
Drains, Box										
Drains, Pipe		lin. ft.								
Cattle Guards		Each.								
Fencing		Mile.								
Road Crossings										
Signs										
Ties		Each.								
Tie-Plates		"								
Steel Rails		Ton.								
Joints Complete		Each.								
Spikes		Keg.								
Laying and Surfacing		Mile.								
Sidings		lin. ft.								
Switch Furniture		Set.								
Railroad Crossings										
Ballast										
Telegraph		Mile.								
Passenger Depot		Each.								
Freight Depot		"								
Cotton Platform		sq. ft.								
Section House		Each.								
Water Station		"								
Stock Pen		"								
Coal Chute		"								
Track Scales		"								
Total										

*Mile No.*_____ *From*_____ *Rail*_____

ESTIMATE SHEET; "B," ROLLING STOCK.
ESTIMATE SHEET; "C," MISCELLANEOUS ITEMS.

RAILROAD COMMISSION OF TEXAS.

Rail

ITEM.	Estimate Upon Which Authority to Issue Stock and Bonds is Based. Made.....190..			Estimate for Registration of Stock and Bonds. Made.....190..			Estimate for Registration of Stock and Bonds. Made.....190..		
	No.	Rate.	Value.	No.	Rate.	Value.	No.	Rate.	Value.
Passenger Locomotives									
Freight Locomotives									
Switch Locomotives									
Passenger Cars									
Chair Cars									
Baggage Cars									
Express Cars									
Postal Cars									
Combination Cars									
Officers' Cars									
Box Cars									
Flat Cars									
Stock Cars									
Coal Cars									
Tank Cars									
Refrigerator Cars									
Water Cars									
Hand Cars									
Push Cars									
Total "B"									
"C" MISCELLANEOUS:									
Road Tools									
B. & B. Tools									
Machine Tools									
Machine Shops									
Machinery									
Round Houses									
Turntables									
Furniture, Etc									
Stationery									
Materials and Supplies									
Terminal Grounds									
Franchises									
Charter Fees, Etc									
Legal and Engr. Expenses									
Interest during Constr									
Total "C"									
Totals "B" and "C."									

Rail

No allowance is made, in the estimates of the Commission, for discount on bonds, brokerage, etc., it being considered, theoretically speaking, that the stocks and bonds represent, dollar for dollar, actual physical values. The margin for any discount on bonds or profits of construction, which the promoters and builders of railroads in Texas have, is therefore confined to the value allowed for the donations made to the railroad by outside interests, and the difference, if there be any on the positive side, between the actual cost of the railroad and its value as the same may be estimated by the Commission.

The emergency clause of the law, given in Articles 4584 b and 4584 f, which permits the Commission to authorize the execution of indebtedness 50% in excess of the actual value of the property, as it may determine the same to be, has been invoked a number of times by the railroads to cover such items as discount of bonds, brokerage, etc., but the Commission has in all cases refused to exercise its prerogative in the matter, holding uniformly that it is the intention of the law that the said emergency provision should be invoked only in cases to cover losses by railroads from extraordinary causes, such as fire, floods, etc.

RULES AND REGULATIONS CONTROLLING THE RAILROAD COMMISSION OF TEXAS IN ISSUANCE OF RAILROAD STOCKS AND BONDS.

"For the information of the public and to secure the protection to investors contemplated by the law governing the issuance and registration of railroad stocks and bonds, the Commission has prescribed the following as the method of procedure under the law necessary to secure its authority to issue and have registered railroad stocks and bonds:

"APPLICATION FOR AUTHORITY TO ISSUE BONDS.

"An application to the Commission for authority to issue bonds must set forth in the body of the application, or as an exhibit thereto, the following facts which must be signed by the president and attested by the secretary of the company making the application:

"1st. Copy of the company's charter.

"2nd. Copy of the published notice to stockholders calling a meeting thereof for the purpose of authorizing the directors of the company to apply to the Commission for authority to issue bonds, and a copy of the resolutions adopted at such meeting of the stockholders authorizing the directors to apply to the Commission for authority to issue bonds.

"3rd. The contract with the construction company proposing to construct the road, if any.

"4th. Alignment map of located road, on a scale of not less than 2 000 feet to the inch, showing property, city and county lines.

"Deeds to right of way and other real estate acquired by the corporation for railroad purposes, duly attested. These must have reference to the maps, and property; city and county lines should be properly tied to the located line of the railroad in the customary manner.

"If there be lands traversed by the located line to which title has not been acquired by the corporation at the time of application, whether by reason of delay consequent upon the necessity of condemnation proceedings, or from other causes satisfactory to the Commission, the deeds thereto shall be filed with the Commission as they may be obtained, and their description noted in place upon the maps.

"Any change that may be made in the located line of the railroad either before or during construction must be reported to the Commission in order that the same be duly recorded, awaiting the final action of the Commission.

"Maps of station grounds, yards and terminals, on a scale of not less than 200 feet to the inch, showing locations of tracks, buildings and other structures, and their relation to the streets, blocks and lots of the town or city in which they are situated, if any.

"Profile of located or completed road or part of road on a scale of not less than 400 feet to the inch horizontal, and 30 feet to the inch vertical, showing the following: Nature of timber on right of way originally, if any; quantities of excavation and embankment and their classification; location and length of sidings and spurs; location of water stations and nature of supply; location and character of bridges, buildings, and all other way structures.

"Plans and specifications for the following: Graduation; ballast and track; bridges, buildings and other standard structures.

"Rolling stock: Number and class of locomotives; number and class of cars of all kinds; miscellaneous equipment.

"Detailed estimate of the cost of the proposed or completed road.

"5th. The extent and value of work already done or in process of completion.

"6th. Amount of money and amount and value of property received as donations to the company.

"7th. Aggregate amount of stock subscribed to the capital of the company and amount paid thereon.

"8th. Any and all other facts showing the value of the franchises and property offered as security for the contemplated issue of bonds.

"The following statement must accompany the application as an exhibit:

"List of subscribers to capital stock, number of shares subscribed by each, amount of stock represented by each share, and amount of

cash, labor or property paid thereon. Opposite each name on said list must be affixed a number beginning with one, or the next highest number of any certificate of stock previously issued. Said statement must be certified to by the president of the board or presiding officer of the meeting at which such stock was authorized, as correct and sign the same in person. Said statement or certificate must be entered upon the minutes of the board, attested by the secretary thereof, with the company seal affixed thereto and deposited with the Railroad Commission. (See Sec. 7 of An Act to Regulate Issuance of Railroad Stocks and Bonds.)

"Upon the filing of such application, and a full compliance with the law, the Commission will issue an order granting the corporation applying therefor authority to issue its bonds to become a first lien on its franchises, property and completed road or part of road, such bonds to be approved and directed to be registered only on completed road, or part of road, to the extent of the value thereof as ascertained by the Commission.

"APPLICATION FOR APPROVAL AND REGISTRATION OF BONDS BY THE COMMISSION.

"Application to the Commission to approve bonds and have them registered in the Secretary of State's office must set forth under oath of the president of the company making the application:

"1st. The amount of stock of said company.

"2nd. The amount of outstanding bonds, if any.

"3rd. The number of miles of completed road or part of road upon which the bonds are to become a first mortgage.

"The bonds presented for approval and registration must be executed in accordance with Section 8 of the act regulating the issuance of the same.

"Upon filing of each application and the presentation of the bonds desired to be approved and registered thereunder the Commission will direct its engineer to inspect the road or part of road completed, and ascertain if it has been constructed and equipped in accordance with the specifications and the profile set out in its application to the Commission for authority to issue bonds. If it finds that said completed road or part of road proposed as security for the bonds presented for approval and registration has been built in good faith and that it is worth the amount for which it is proposed as security, an order will be issued to the Secretary of State directing him to register bonds to the extent of the value of said completed road or part of road.

"No bonds will hereafter be approved and directed to be registered by the Secretary of State except upon roads or parts of roads completed and equipped, and then only to the amount of the value thereof as determined after investigation by the Commission.

" REGISTRATION OF STOCKS.

" The issuance of certificates of railroad stock to subscribers to the capital stock must be in accordance with Section 7 of the act regulating the issuance of railroad stocks and bonds and must be only on completed road or part of road. When so issued the Commission will direct their registration in the office of the Secretary of State.

" The amount of the stocks and bonds together in the aggregate must not exceed the reasonable value of the property of the company issuing them."

RAILROADS WHICH HAVE BEEN VALUED BY THE COMMISSION.

Since the passage of the Stock and Bond Law, the Railroad Commission has, from time to time, valued the railroads of Texas, and filed its valuations with the Secretary of State. Table No. 1 is a list of the railroads which have been valued, up to October 20th, 1903, together with their valuations, as the same will appear in the Twelfth Annual Report of the Railroad Commission, 1903.

EFFECT OF THE STOCK AND BOND LAW.

In connection with the foregoing statement of the Railroad Stock and Bond Law of Texas and the description of the methods used by the Railroad Commission when valuing railroads, under its provisions, it will no doubt be of interest to mention what the general effect of the law has been on the railroad securities of the State. Table No. 2, which will appear in the Twelfth Annual Report of the Railroad Commission of Texas, 1903, very briefly, but effectually, shows the principal effect of the Stock and Bond Law, which has been to reduce steadily the average outstanding indebtedness of the railroads of Texas, per mile, and this has been done in the face of an actual increase in physical values and an increase in net earnings of the properties.

When it is considered that all indebtedness of the railroads outstanding at the time of the passage of the Stock and Bond Law remained in effect, and is still in effect, it will be seen that the reduction in the average indebtedness per mile of the railroads of Texas has been due largely to the restriction which the law placed on the issuance of securities for the 1 875 miles of new railroad which

have been constructed since the passage of the law, and such old railroad as was sold out, at receiver's, or other sale, and reorganized.

TABLE No. 1.

Name of railroad.	Mileage.	Valuation.	Value per mile.
Beaumont Wharf and Terminal	3.08	$179 481	$58 273
Cane Belt	68.69	830 968	12 097
Chicago, Rock Island and Mexico	91.75	1 475 361	16 080
Chicago, Rock Island and Texas	147.29	2 760 423	18 741
Dallas, Cleburne and Southwestern	9.77	155 917	15 959
Denison, Bonham and New Orleans	24.11	375 266	15 565
Eastern Texas	30.41	454 528	14 947
Fort Worth and Denver City	454.13	5 771 582	12 709
Fort Worth and Rio Grande	148.10	2 003 839	14 003
Galveston, Harrisburg and San Antonio	919.06	16 142 298	17 564
Galveston, Houston and Henderson	48.85	1 527 023	31 259
Galveston, Houston and Northern	56.36	1 000 000	17 743
Gulf, Beaumont and Kansas City	67.55	676 485	10 015
Gulf, Beaumont and Great Northern	58.22	853 123	14 653
Gulf, Colorado and Santa Fé	987.34	16 649 746	16 863
Gulf, Western Texas and Pacific	109.67	1 318 082	12 019
Hearne and Brazos Valley	16.42	106 624	6 493
Houston and Texas Central	690.42	13 683 834	19 820
Houston, East and West Texas	190.69	2 042 693	10 712
International and Great Northern	1 049.93	19 928 547	18 981
Missouri, Kansas and Texas, of Texas	1 069.83	16 688 500	15 599
New York, Texas and Mexican	122.93	1 464 206	11 911
Orange and Northwestern	32.80	458 793	13 987
Paris and Great Northern	16.18	288 718	17 844
Pecos River	54.13	392 405	7 249
Red River, Texas and Southern	57.49	1 215 332	21 140
Rio Grande	22.17	310 551	14 008
Rio Grande and Eagle Pass	26.89	234 695	8 729
Rio Grande and El Paso	20.15	481 824	23 912
St. Louis, San Francisco and Texas	6.74	407 454	60 453
St. Louis Southwestern, of Texas	652.51	10 057 889	15 706
San Antonio and Aransas Pass	687.67	8 677 698	12 619
Southern Kansas, of Texas	114.95	1 190 672	10 358
Sugarland	14.12	109 415	7 749
Texarkana and Fort Smith	79.24	1 603 789	20 240
Texas and Louisiana	21.91	203 688	9 297
Texas and New Orleans	433.14	8 667 796	18 896
Texas and Pacific	1 089.33	17 730 689	17 060
Texas Central	215.36	2 885 256	13 397
Texas Mexican	162.40	1 457 638	8 976
Texas Midland	112.49	1 914 578	17 020
Texas, Sabine Valley and Northwestern	38.41	398 848	10 384
Texas Southern	68.69	743 980	10 831
Velasco, Brazos and Northern	20.06	240 656	11 997
Weatherford, Mineral Wells and Northwestern	23.00	327 496	14 239
Wichita Falls	17.96	242 838	13 521
Wichita Valley	51.36	486 768	9 478
Total	10 348.75	$166 817 992	$16 120

This reduction in the average indebtedness per mile of the railroads of Texas will continue as long as the Stock and Bond Law remains in effect, and until their capitalization approaches the physical value of the properties, as it may be determined by the Railroad Commission.

TABLE No. 2.

On June 30th.	Miles of railway in operation.	Stocks outstanding, per mile.	Bonds outstanding, per mile.	Total stocks and bonds outstanding, per mile.
1894	9 154	$15 076	$25 726	$40 802
1895	9 291	14 874	25 420	40 294
1896	9 437	14 647	25 302	39 949
1897	9 484	14 320	24 793	39 113
1898	9 540	14 205	24 036	38 241
1899	9 702	13 997	23 562	37 559
1900	9 867	13 724	23 202	36 926
1901	10 154	12 922	22 649	35 571
1902	10 617	12 388	21 779	34 167
1903	11 029	11 971	21 464	33 435
Total reduction for 9 years		$3 105	$4 262	$7 367
Average reduction per year for 9 years		$345.00	$473.55	$818.55

DISCUSSION.

Mr. Corthell. E. L. CORTHELL, M. AM. SOC. C. E. (by letter).—The writer, since 1887, has had some knowledge of the facts referred to in this paper, has been familiar with some Texas railroads and their terminals, and is acquainted, more or less, with the physical conditions of the southern and western portions of that State.

He has no doubt whatever of the general beneficence of the Texas railroad law, beneficent generally to all parties except perhaps the old-time railroad promoter.

The law, and generally its just operation, has cured many unmitigated and notorious evils. Not only has the public in Texas been benefited, but also the investor in railroad securities from outside the State. The people of Texas now have just and uniform rates of transportation, and the investor knows what he is purchasing, and may be reasonably sure of a return on his investment.

In the methods of valuation described, and these are about the only strictly engineering questions involved in the paper, there appear to be some serious objections.

Both old railroads—that is, those which existed prior to the passage of a law in 1894—and new roads, are valued according to the judgment of the State Engineer, "allowing current market prices for labor."

Would it not be more reasonable to obtain the contract prices paid by the company in constructing its railroads, and the actual cost from its books when doing the work by its own day labor? Are the railroad officials so untrustworthy and deceitful that they could not be trusted to give correct figures? One would hardly expect that the officers of a railroad company would enter into collusion with the contractors in order to show a larger cost than the actual; if so, then the law could reach them and punish them for fraudulent practice.

In valuing pre-existing properties, the author states that "no additional allowance was made for the value of seasoned roadbed, etc." A most important item in the real cost of an old road is thus eliminated from the valuation, without further reference to it or explanation, while every railroad engineer who has had the experience in maintaining railroads knows how costly "seasoning" is. As an illustration, it may be stated, from reliable reports, that, considering all the various lines of the Southern Pacific System in Texas, for the ten years prior to 1894, the cost of "seasoning," that is, physical betterments, changes in alignment, restoring washed banks, raising roadbed, rebuilding after washouts (a common experience in Texas), raising track, etc., etc., cost $4 734.30 per mile of track. This appears to be all the more important when considering the extremely low val-

nation which the Commission has placed on the road. Referring to Table No. 1, it is found, for instance, that the 919.06 miles of the "Galveston, Harrisburg and San Antonio" (one of the principal lines of the main line of the Southern Pacific) are valued at $16 142 298, or only $17 564 per mile, whereas the "seasoning" alone has cost one-fourth of this. Mr. Corthell.

Again, the cost of "legal and engineering expenses and superintendence" is allowed at the rate of 5 to 6% of the total estimated value of the road, which is entirely too low to cover these important and costly items; and where are the other costs, such as contingent expenses (which an engineer would generally estimate at from 15 to 20% of the estimated cost), general expenses, right-of-way agents, and all the multitudinous employees who must be upon every railroad of any importance during its preliminary and constructive periods? And yet, the estimate on which to base the permitted issue of stocks and bonds is made upon the report of an engineer who goes over the road with a profile and estimates his quantities from center heights. Any expert engineer, who has had to examine and report upon the value of railroad properties, knows how utterly inadequate such a method is, to get at the actual cost of a railroad, even approximately. This, the writer knows by his own experience in examining and reporting upon about 600 miles of railroad in this same State of Texas.

Considering all the items that go to make up the actual cost of a good railroad, with all its appurtenances, from the right of way to the rolling stock, the valuation of $15 000 to $20 000 per mile is entirely inadequate to cover it, in Texas, at least.

All the old roads, that is, all roads existing before the Commission made its valuations, are and probably always will be, debarred from the issuance of any more stocks or bonds, no matter how many and how costly may be the physical improvements or equipment they put into their properties, or wish to put into them. Although the valuation of the "Galveston, Harrisburg and San Antonio" is given as $16 142 298, this road, at the time of its construction, had a stock and bond issue of $52 612 400, divided nearly equally between stock and bonds.

No dividends have ever been earned on the stock, and, no doubt, most of it was what may be called watered stock; but until, by betterments and general additions to the value of the property, its valuation by the Commission has reached, on the same mileage, $52 612 400, no stocks or bonds can be issued—a rather hopeless future—so that, to make any increase in equipment or any substantial improvements, there is no method by which the road can borrow any money.

The writer believes that all the companies of old Texas railroads

Mr. Corthell. would have been perfectly satisfied with the work of the Commission, in respect to this feature, had a fair estimate of their properties been made at the outset, so that they might earn the legal rate of the law on their investment, *viz.*, 6 per cent. This refers to the fact that the Commission fixes the rates so that the companies may earn the legal rate of 6% on the valuation of the properties as made by the Commission.

Texas is a great State and a rich agricultural country, and it has a great future. No doubt, any hardship, which unreasonable methods have brought about, will disappear in time, with juster methods of valuation, and a better appreciation of the work which the railroads are compelled to do, and the expenses they have to meet.

The legislators who framed the law and the Commission which enforces it, were actuated by the best of motives and for the general good of all interested, especially the public, which suffered under the former conditions. The writer desires to say that, from a personal acquaintance with the honored Chairman of the Commission from the outset, he knows of no more conscientious and patriotic man than Judge Reagan.

Mr. Lavis. F. Lavis, Assoc. M. Am. Soc. C. E.—As to the low valuation of some of the roads mentioned by Mr. Corthell, the speaker's experience, based on very close estimates of some projected lines in Northern Texas, estimates based on actual final location lines, and very careful examination of the ground, leads him to believe that Mr. Corthell's statement, that the valuations of the Commission were apt to be low, is correct.

The Texas State Railroad Commission, while not, perhaps, absolutely infallible, is, the speaker believes, one of the most efficient in the country. Texas has led all the other States, in miles of new railroad built, for several years past, and it is fortunate for the railroads, as well as for the State, that the laws relating to the railroads, and the administration of them, have been so efficient, and notwithstanding the remarks, as to the courage of promoters of railroad lines in Texas, all the older roads are seizing every opportunity to establish themselves in advantageous locations.

The general character of the lines built in recent years shows that there is a marked departure from the older methods of location and construction. Where possible, with any reasonable, or even considerable, expenditure, the maximum grades are not greater than 0.5 or 0.6%, compensated for curvature, which latter seldom exceeds 4 degrees.

A recent law compels all existing steam railroads to install an interlocking plant at all grade crossings with other steam roads, the expense, both of installation and maintenance, to be borne equally by the two roads. Where a new road, however, proposes to cross a

road already existing, the expense has to be borne entirely by the new road, provided the Railroad Commission decides that a separated crossing at the point in question is not practical, and it is somewhat difficult to demonstrate this to the present Commission. New railroads however, can well afford to spend as much as from $25 000 to $30 000 to avoid the expense of the interlocking plant and its maintenance. Mr. Lavis.

W. H. COVERDALE, ASSOC. M. AM. SOC. C. E.—Although it may seem to be a far cry from the recent tragic death of a London promoter and the no less pathetic situation now confronting the United States Shipbuilding Company to the subject-matter of this paper, yet the fact is that these seemingly diverse matters are but different effects of the mushroom growth which is called over-capitalization. On one hand, there is unrestricted and perhaps unlawful development, resulting in ruin; on the other hand, there is so rigorous a use of the pruning knife that the normal growth is stunted. Mr. Coverdale.

The very first sentence in Mr. Greene's little book on "Corporation Finance" is that a business man or firm must borrow money. With the individual, as with small firms and manufacturers in many lines, the loan may take the form of commercial paper; with the railroad, it generally consists of a mortgage. The regulation of the amount and character of such mortgage is one of the most important matters upon which definite legislation is needed.

Those engineers who have been identified with railroads are doubtless aware that within the last fifty years railroad commissions have been established in various States which deal, but by no means exclusively, with this matter.

These commissions now exist in New York, Massachusetts, Maine, New Hampshire, Vermont, Connecticut, Rhode Island, Pennsylvania, Ohio, Kentucky, Michigan, Wisconsin, Minnesota, North and South Dakota, Nebraska, Illinois, Iowa, Missouri, Texas, Arkansas, California, Virginia, Georgia, North and South Carolina, Tennessee, Alabama, Mississippi, Louisiana and Florida.

Seventeen States have no railroad commissions. Many of these commissions are mandatory, but some are merely advisory, and, indeed, the latter, in some instances, have shown themselves quite as proficient as the mandatory commissions holding greater legal power.

The Massachusetts Commission, which is advisory, only, has developed along broad lines, and its efficiency is well known. The Texas Commission, whose methods are under discussion, is mandatory, and is also well and favorably known. All these commissions deal with questions of public utility and convenience; questions of rates and traffic; questions of accounts, of arbitration, and in some cases of capitalization.

In addition to the State commissions there has been, since 1887,

Mr. Coverdale.

an Interstate Commerce Commission. This Commission devotes much time to rates and tariffs, and has done much to better conditions by doing away with pooling agreements and abolishing the system of secret rebates. Its functions, however, have been limited, and in some cases curtailed, by Supreme Court decisions relative to the intent of the Legislative Act under which it operates, and the main question of absolute control of railroad capitalization is, therefore, broadly speaking, untouched.

When it is considered that there are some forty-nine State governments exercising authority over the railroads of this country, the folly of looking to such diverse interests for harmonious and concerted action is seen. When it is considered, further, that one railroad property may extend over several States, and be subject to a new set of regulations and laws every time its tracks cross a State line, something is seen of the inconvenience and danger, even, of the existing conditions. The word "danger" is used because a multiplicity of conflicting laws means an observance of none. When it is realized that only a portion of the traffic of a railroad can be controlled by State legislation, namely, the business originating and terminating within the State's borders, and that even the import and export traffic on through billing to and from interior points is not affected by the rulings of the Interstate Commerce Commission, it is seen that the present methods of control are inadequate, and that evasions of the laws may be made with ease.

The present conditions, although in many respects unsatisfactory, mark substantial progress over those obtaining a few decades ago.

The years from 1850 to the beginning of the War marked the rapid growth of railroads in the Middle West, and the consolidation into trunk lines of the various smaller companies throughout the more thickly populated Eastern States. In those days the railroad was, in name and in fact, a private corporation operating without restrictions as to rates and methods. To-day the railroad is a public-service corporation, and as such is in a measure subject to Government restrictions as to rates of traffic, public convenience, and publicity of operation.

In 1868 to 1872, during which years many of the western roads were constructed, the over-capitalization of these new lines was such that, notwithstanding Government land grants and money bonuses, disaster was not averted in the panic of '73.

The lesson learned then has not been forgotten, and the conservatism of to-day is substantial in character when compared with past conditions.

Disregarding the possibility of Government ownership or control of railroads, as too radical a departure from present conditions, and considering only Government supervision in all matters, including

traffic rates and capitalization; and proceeding, also, upon the basis of rates which competitive traffic will bear, the question may now be asked: "Wherein consists the security of a railroad mortgage, and what assets may with propriety be considered as collateral therefor?" Mr. Coverdale.

In answering this question it is necessary to keep in mind the essential difference between a mortgage on real estate or on a manufacturing plant and a mortgage on railroad property. Land has a real value which may readily be ascertained, and its value as a mortgage security is that percentage on its market value which can be obtained for it at a forced sale; a manufacturing concern has also real value in such assets or real estate, plant, stock and finished product, and such a property should wreck for more than the amount of its outstanding obligations.

A railroad, it is true, has some real assets in the way of land, roadbed, buildings, and rolling stock, but it produces from this property nothing but that intangible, highly perishable commodity called transportation; it must operate even at a loss to prevent utter ruin; it cannot be devoted to other uses; it cannot be sold under foreclosure because its use is so restricted, and purchasers, therefore, few, but must generally be reorganized upon a more conservative basis, which means a loss to security holders.

A railroad mortgage, therefore, is secured only in part by its physical assets, notwithstanding the legal phraseology on the bond, and in part by its franchises and special privileges which contribute to its earning power no less than do its rolling stock and track.

Inasmuch as an *ex post facto* law has never been in general favor, it seems improbable that a readjustment in capitalization of existing roads could be made by Government authority, except at times of reorganization. At such times, with full operating statistics available, a fair valuation could be made in the following manner: Take the average yearly gross earnings and deduct therefrom the average operating expenses plus a proper amount for annual maintenance, taxes and betterments; then, as a reserve fund, set aside such amount as will in the course of a few years provide a surplus against fluctuations of business, and the balance, capitalized, will represent the amount of securities which the property is worth.

By deducting from this sum the physical cost or value of the property, the proper value of the franchises, good will, or earning capacity is secured.

For example, take a railroad earning a gross income of $10 000 000 per year. Such a road in Texas would be 2 000 miles long, as the average gross earnings of railroads in that State are about $5 000 per mile. Deduct now 70% for operation, maintenance and taxes, which, again, fairly represents actual conditions, and there remains a balance of $3 000 000.

Mr. Coverdale.

Out of this sum set aside 5% of the gross earnings, or $500 000, for reserve fund, and there is a balance of $2 500 000.

Capitalizing this balance at 5% gives a total of $50 000 000, which represents the total value of the property. The average cost of construction and equipment per mile, as determined by the Texas Commission, is about $16 000, so that the physical value of the railroad would be $32 000 000, leaving an actual value of $18 000 000 to be ascribed to franchises and earning power.

This total valuation would be at the rate of $25 000 per mile, as against $40 802 per mile, which was the average in Texas in 1894, just prior to the act under which the Commission was created; and, as against $16 120 per mile, which is the average for all the Texas railroads which the Commission has valued.

Such a method is conservative when compared with uncontrolled capitalization, and it approximates actual conditions more nearly than the restricted method which considers physical values only.

The foregoing figures represent both bonds and stock, and for two reasons: The first is that the Texas Commission figures include both classes of securities; and the second is that railroad stocks should have an investment value rather than a speculative value.

In the case of new roads, where no earning statistics are available, these could be estimated, on a conservative basis, from adjacent roads, or they could be omitted entirely and a temporary or partial valuation made on the cost of construction. Such valuation should be supplemented on the more liberal basis as soon as the earning capacity of the road is demonstrated; and in valuations of both old and new roads provision should be made for increase of capitalization commensurate with growth of earnings.

The speaker has spent considerable time in Texas on railroad work, and is acquainted, superficially, with the methods of the Commission in the matter of valuations.

He had the pleasure of being on a railroad inspection trip in 1902 with the author, and was much impressed with the thoroughness and care evinced, even in matters of small detail. The Texas Commission has had a wholesome effect on Texas railroading, and is popular both with the railroads and the public.

In closing, the speaker wishes to note three pertinent items:

First.—The Texas railroads constructed prior to the passage of the act of 1893 seem to have an advantage over those built since, as their capitalization is unaffected. Perhaps this is a fallacy, however, as a very considerable percentage of them was in the hands of receivers when the law took effect.

Second.—Railroads doing an interstate business are not affected in the matter of capitalization, as are those entirely in the State, for the reason that they can still issue blanket mortgages covering the whole property irrespective of the Texas law.

Third.—The practice of the Commission, in allowing first cost of construction without depreciation, seems to favor unduly those roads which are not maintained properly, and there are several such in Texas. Mr. Coverdale.

W. D. TAYLOR, M. AM. SOC. C. E. (by letter).—Contrary to former custom, the business of valuing railroad property is falling more and more into the hands of engineers. The most frequent railway appraisals that have been made in the past have been made for some other railway for the purpose of determining whether or not it was desirable for the road in question to try to acquire control of the road appraised. The basis of such appraisals was the actual past or prospective earning power; and, in making such appraisals, the engineer and the account of the physical properties came in for small consideration. Mr. Taylor.

Engineering literature, on the subject of scientific railway appraisal, is scanty and unsatisfactory. The results of most of such appraisals as have been made in the past—those made by one railway expert for another—have not found their way into print; and, as far as they would bear upon what are called the pure engineering questions involved in such appraisals, they would possess small value if they had.

With the exception of Mr. Thompson's paper, the only helpful literature bearing on the subject the writer has been able to find, is the account, in various journals, of the appraisal, made in 1900–01 by the State Board of Assessors of Michigan, of all the railways of that State. An interesting account of this appraisal is also given in the report of the Board of State Tax Commissioners of Michigan for 1902. The appraisal of the physical properties of the railways of the State was made under the supervision of Professor M. E. Cooley, of the University of Michigan. The appraisal of the railways of the State, as a whole, including both the value of the physical properties and the value of the earning power, or intangible value, was made by Professor Henry C. Adams, also of the University of Michigan, and Statistician of the Interstate Commerce Commission. This appraisal was made for purposes of taxation.

The writer has been employed since June, 1903, by the State Board of Assessment of Wisconsin, in making an appraisal, similar to that made by Professor Cooley in Michigan, of all the railroads of Wisconsin, and for a similar purpose. An account of the work in Wisconsin and the plan adopted for finding the value of the physical properties of the railroads can be found in *Engineering News* of March 31st, 1904.

There are few kinds of engineering work which introduce more intricate and puzzling problems than the appraisal of a large railway system. Many of these problems are economic rather than ques-

Mr. Taylor. tions of pure engineering, and, as in much other work now committed to engineers, it is difficult to tell where engineering ends and economics begin.

In railway appraisals the standpoint assumed, or basis of appraisal, is all-important. Two engineers may undertake the appraisal of the same property, use methods which seem perfectly fair to each, and arrive at results differing as much as 20 or 30% in case the basis of appraisal used is not the same. Instances can be cited where the discrepancies were as great as this.

To outline the standpoint, or scheme of appraisal, the following points, among others, must be considered and decided beforehand:

1.—As to whether the earning power or the value of the physical properties of the road is to form the basis of the appraisal; and, whichever one of these is decided upon as forming this basis, how, and to what extent, is the final valuation to be modified by the other?

2.—In arriving at the value of the physical properties:

Shall the cost of reproduction of the physical properties with all property new, or with all property in the existing deteriorated condition, be used?

Shall the market prices of railway goods and material be used for a certain year, or the average for a period of years?

Shall present-day prices and conditions be held as controlling, or shall prices and conditions existing at the time of the construction of the road control?

Shall the land held by the road for right of way and terminals be considered as having the same value as the market value, for other purposes, of contiguous land, or shall the value placed upon this land include the amounts generally paid for real and imaginary damages to contiguous property?

If the cost of reproduction of the physical properties in the existing deteriorated condition is to be used, shall there be established a principle of minimum values for all classes of property and structures in actual use, such as that established by the railways among themselves to govern the amount of payment for cars belonging to another system destroyed in wrecks; or shall the actual value of the property at the time of the appraisal be taken, even though that value be zero, as in the case of a wooden bridge or station building about to be renewed?

What percentages of the value of the usual known construction items shall be used to cover such uncertain items as "engineering and superintendence," "freight on construction material," "interest during construction," "organization and legal expenses," and "contingencies?"

Mr. Taylor.

In case only a part of a railway, or of a system, is being appraised, shall the equipment be prorated on a basis of the time it occupies the tracks appraised, or on a basis of earnings, track-mileage, ton-mileage, or car mileage?

The purpose of the appraisal must decide many of these points. On the supposition that a railway property is bought and sold like other property, on a basis of total value (which supposition, of course, is contrary to fact), all the problems and conflicting conditions in the appraisal of railways will probably arise in making appraisals for the following five causes:

1.—For total valuation, as for sale or purchase;
2.—For joint rights in operation;
3.—For determining permissible rates;
4.—For regulating the issue of securities, as in Texas;
5.—For assessment in taxation, as in Michigan and Wisconsin.

It should be noticed that it depends very much upon the purpose of an appraisal as to whether or not the selfish interest of the owners leans toward a high or low valuation. In an appraisal for Causes 1 and 2, the interest of one road or company will lean toward a high, and of the other toward a low, valuation. In appraisals made for Causes 3 and 4, the selfish interest of the owners leans toward the highest valuation possible, and, in an appraisal for Cause 5, the same interest leans toward the lowest valuation possible.

In the past, the writer has had railway appraisals to make for Causes 2 and 3, and, as already noted, is now engaged on railway appraisals for Cause 5. As a result of this experience he has found it necessary to modify considerably some preconceived views on the subject.

To appraise a piece of property, as a farm, means to find its value. And the value of the farm, in theory at least, is the same, no matter what the purpose of the appraisal, no matter whether the appraisal is made for an intending purchaser, to establish its owner's credit, or to determine his assessment for taxation.

The same principle, of a fixed value, cannot rightly be deemed to hold for a railway. The value of a railway need not necessarily be the same when appraised for assessment in taxation and when appraised to regulate the issue of securities. This difference is recognized by the standpoint assumed in the Michigan and Wisconsin appraisals, on the one hand, and by the Texas appraisal, as explained by the author, on the other. In the last the finding of the cost of reproduction, with every part and parcel of the physical properties in new condition, is the object of the appraisal. In the other two, the cost of reproducing the physical properties in their actual existing deteriorated condition is made the basis of value of the whole property.

In appraisals made for Causes 3 and 4, there are strong reasons

Mr. Taylor. for believing that the cost of all old roadbeds and structures which have been abandoned in the process of improvement should be included in the appraisal. Unless the expenditures for such betterments are paid out of the earnings of the road, the road's fixed charges should usually be loaded up with the interest on the first cost of the abandoned work. It might even be claimed with justice that the cost of many expensive mistakes and accidents should be included in an appraisal for these purposes. In an appraisal made by the writer for Cause 3, a spur line had been built to a mine on what was, apparently, judging from the conformation of the country as developed by the surveys, the best line. However, the soil over this route proved so treacherous that it was found impossible to keep the track up, and the line was rebuilt on an entirely different and more expensive route. In the writer's appraisal, the cost of constructing both of these lines was included, because the fixed charges of the road were loaded with the interest on the cost of both; but, in an appraisal for Cause 5, the cost of constructing the abandoned lines and structures certainly should have no place.

The author states that in the Texas appraisal no deductions were made for depreciation in the value of materials on account of wear and use, because it was considered that the roads were bound to keep their property in first-class condition. This will hardly hold, because actual careful appraisals show that even the best roads do not keep their property in a condition which warrants the placing of the value of their physical properties at much more than 85% of the cost of those properties new. A better reason for this standpoint, however, is that the cost of reproduction, with all properties new, would represent, in appraisals for Causes 3 and 4, the outlay necessary for the company to make in order to begin its present business. This would be true, except in so far as the road under appraisal bought second-hand material, as cars and rails, in the first place.

In an appraisal for Cause 2, the conceived cost of reproduction of the physical properties in existing condition should govern. That is, the appraisal should include only the physical properties affected, and allowance for the effect of wear, time, and use, should be made.

In an appraisal made for Causes 1 and 5, it appears that the standpoint assumed should be precisely the same. In this statement it is supposed, of course, as above, that in the purchase of a railway the full value of its properties and its business is to be paid. In arriving at the value of the property as a whole, both the intangible value and the cost of reproducing the physical properties in their existing deteriorated condition should be considered.

The writer considers Mr. Corthell's estimate of the contingencies to be allowed in making an appraisal of a completed road exceedingly high; but he does not believe that the contingency item should be

Mr. Taylor.

neglected, as is done in the Texas appraisal. The chief engineers of the three largest roads in Wisconsin have contended that, in an estimate of a completed road, a contingency account should have no place. In the Michigan appraisal, Mr. Cooley used a contingency allowance of 10%; and the writer, in the Wisconsin appraisal, 5.5 per cent. Two chief engineers of roads in Minnesota, who made in 1901 an estimate of the physical properties of their own roads, in an appraisal for the adjustment of rates, in which case these engineers might be expected to use as high a contingency percentage as they could consistently, made a contingency allowance of only 5 per cent.

The writer agrees with Mr. Corthell in the belief that the method that the author is using in Texas gives a low value; but he does not believe that, in making an estimate of the value of the physical properties, the value of the "seasoning" of the roadbed can be computed so as to make very much difference in the final estimate. Still, it is undoubtedly true that the operating value of a seasoned roadbed is very great. However, only the grading is affected beneficially by the seasoning. In Mr. Cooley's expert valuation of the properties and 10 900 miles of track of the Michigan roads, he found the cost of reproduction, with all property new, $204 191 000, and the cost of all the grading $21 700 000. Thus, in Michigan only 10.6% of the cost of reproduction of the physical properties would be affected by this seasoning. In the Wisconsin appraisal the writer has omitted the item, on account of its uncertainty, but in other appraisals he valued it at from $450 to $500 a mile for roads which had been completed from eight to fifteen years.

It seems possible that the fact that the actual cost of grading an ordinary railroad is such a small part of the cost of the road as a whole would justify the author's method of using the center heights only in calculating the grading quantities, although Mr. Corthell holds this method to be inexact. An occasional mile of road may be found on hillside work where this method will give very inaccurate results; but, as known by every engineer who has made preliminary estimates by this method and then verified his work by actual construction with quantities calculated accurately, this method gives results surprisingly accurate in ordinary country. If the grading is of the same relative importance in Texas as in Michigan, it would require an error of 10% in the grading quantities to affect the value of the physical properties by 1 per cent.

An item in the author's method which tends toward a very low valuation for the Texas appraisal is that of right of way. The author says that the values applied to the right of way were in accordance with the current market value of the adjoining property. This may give very fair results across the Texas plains and grazing lands, where much of the right of way is donated or sold at a nominal value as

Mr. Taylor. an inducement to build; but it is certainly very much below the figures the railways actually have to pay in thickly settled farming districts, or in crossing suburban or residence property in entering cities. In such cases the railways not only have to pay the very highest market values for the land actually taken, but they have to pay large amounts to cover real or supposed damages to contiguous property. In crossing a farmer's land a road will have to pay for inconveniences imposed upon the farmer in the way of smoke, noise, danger and interference with the drainage of his land and its accessibility. Even cutting off the view of a part of a farmer's land from his house by a high embankment has been known to more than double the cost of the land actually used. In this way the right-of-way value is usually very much greater than the market value of the land for other purposes.

The information obtained from both the Michigan and Wisconsin appraisals would go to show that the ratio of the right-of-way value to the market value of the land for other purposes is greater for good farming lands than for any other kind. The writer made a thorough investigation in Wisconsin to determine what this right-of-way ratio had actually been in the case of the three most important pieces of railway construction done in the State in recent years. All these lines were constructed in 1900–01. One was constructed in the southeastern part of the State, in good farming lands, in a locality already well supplied with railroad facilities. The next was in the northwest central portion of the State, where the land was owned in larger areas, and was not nearly as valuable. The third was in the east central portion of the State, and the land was very variable.

Portions of the last two lines ran through country already well supplied with railroad facilities, and portions were in new country. The disposition of the land owners toward the construction of the road was probably generally favorable in the last two cases and unfavorable in the first. Quotations are here made from the circular letters announcing the result of the investigations to the several railroads of the State.

The first investigation covered 32 miles of line.

"The direction in which the road runs, northwest and southeast, necessitated cutting into the farms at inconvenient angles.

"In the investigation, all transfers not *bona fide* sales, such as where the consideration was $1.00, were excluded. Village right-of-way land was also excluded, and some cases where excessive rates per acre were paid for very small corners of land were excluded. * * * We have seventy-four *bona fide* sales along the line of road from one land owner to another during the five-year period ending June 30th, 1902, which included 5 604.5 acres of farming land, at a total cost of $304 323, giving an average price per acre along the line of road for farming lands of $54.30. One hundred and sixteen *bona fide* sales of right of way to the railway included 382.57 acres at a total cost of $108 614.07; which gives an average price of right-of-way land through the two

counties of $283.90. Thus the road paid by ordinary purchase an average of 522% of the market value of the farming land for its right-of-way land." Mr. Taylor.

The second investigation covered 35 miles of line.

"Forty-one land sales, in the said five-year period, contiguous to the line of the road, included the transfer of 10 775 acres, at a cost of $108 927.01, making an average price per acre of $10.11. The largest transfer in the above was 738 acres, and there were ten of the forty-one transfers which included 280 acres or more. Twenty-six transfers of right of way to the railway company included 247.14 acres at a cost of $10 166.50, making an average price per acre of $41.14.

"Thus the ratio of the right-of-way value to the market value for the right of way actually purchased * * * was 407 per cent."

The third investigation covered 82 miles of line.

"Four hundred and eighty-eight *bona fide* sales of farming land, during the five-year period, contiguous to the road, included the transfer of 39 155.7 acres, at a total cost of $452 309.70, giving an average price per acre of $11.55. Two hundred and thirty-nine right-of-way sales included 971.37 acres at a total cost of $35 493.13, giving an average price per acre for the right of way purchased in *bona fide* sales of $36.54. This gives a right-of-way ratio for the whole line * * * of 316 per cent."

It should be stated here that at a hearing, before the State Board of Assessment, of various representatives of the railroad companies of the State in January, 1904, it was claimed by the railroads that the results of an investigation made by them showed that the property owners of the State, knowing that the assessment of their property for taxation was made up largely from data furnished by an examination of the records of sales, had generally fallen into the habit of recording the consideration in sales at amounts much below those actually paid. Evidence was brought forward tending to support the conclusion that the records of sales throughout the State would show a land value of not more than two-thirds of the real value. If that contention is correct, the foregoing ratios should be changed from 522, 407, and 316% to 348, 272, and 210%, respectively.

It is regrettable that these ratios for city property held for right of way and terminals could not be investigated satisfactorily, because no road in the State has built a line in recent years into any important city through which that road did not already have a line. As to the ratios which were fair to the roads in valuing city property, the writer was compelled to rely upon the results of his own experience in construction, and upon such outside information as was available.

In the Wisconsin appraisal, the method followed for valuing the right of way and terminal lands was about as given below. Parts of the right of way of some of the larger systems are estimated at higher ratios than this, but in such cases the roads themselves fixed the right-of-way value.

The market value for other purposes of the right of way and terminal lands was judged to be the same as that of contiguous property.

Mr. Taylor. In farming lands, small towns, and suburban and residence property, the right-of-way value was taken to be 250% of the market value for other purposes.

In city property, the right-of-way value was taken to be 133% of the market value for other purposes, where the land was owned in strips of 100 ft. width or less, and 110% of the market value for other purposes, where the land was owned in blocks, or in widths greater than 100 ft.

Competent comprehensive railway appraisals will probably be required often in the future. On account of the increasing demand for publicity in the accounts and in regard to the conditions of public service corporations, and, on account of the increasing suspicion in the popular mind that the railways are disposed to make extortionate charges through their ability to make combinations adverse to the interest of their patrons, it seems inevitable that railway appraisals for regulating rates will be common in the future. It is plain that, where a State undertakes to regulate the issue of securities, it will be necessary to make appraisals from time to time while the law endures, as values fluctuate and mileage increases. Similarly, appraisals will be needed from time to time where a State adopts the so-called *ad valorem* method of taxation, as is now the case in Michigan and Wisconsin. It seems probable, in this day of scientific accounting, that the old method of appraising a railway, which consisted in guessing at the value per mile of road through county or state, cannot long survive.

Since, then, it is probable that railway appraisals are so often to be required, and for such various purposes, and since the result of the appraisal is so dependent upon the method used and the basis adopted, it seems desirable that the whole matter should be discussed in all its bearings by engineers, railway managers and those interested in railway economics until the best and fairest methods can be decided. The author presents a very timely paper, offering opportunity for such a discussion.

Mr. Thompson. R. A. THOMPSON, ASSOC. M. AM. SOC. C. E. (by letter.)—The writer, while admitting that grave objections may be raised to the methods used by the Railroad Commission of Texas in valuing railroad properties under the provisions of the Stock and Bond Law, believes that other objections, some perhaps more serious, might be raised against any other system of valuation that could be formulated, and that it is probable that the Texas Law, when administered fairly and impartially, is as fair and just, both to the railroads and the public, as it is possible to provide.

As the law now stands, it is susceptible of considerable freedom of application, and, under such limitations as "to promote public interests and protect private rights" and "public interests and the

preservation of the property," no one will maintain that a fair and reasonable tribunal has not ample opportunity to mete out just and liberal treatment to the railroads, and, at the same time, give the people the protection contemplated by the law. It is left absolutely to the discretion of the Railroad Commission to authorize the issuance of stock and bonds 50% in excess of the value of the railroad property. Mr. Thompson.

That the law and the methods used by the Commission in valuing railroad property have not entirely discouraged railroad building in Texas is shown by the fact that since the passage of the law more than 2 000 miles of railroad have been built, both by trunk lines and by small independent corporations, and have been paid for by securities issued under its restrictions. This is evidence that the law does not hamper or entirely thwart legitimate railroad construction. The hue and cry raised against the law and its administration can in every case be traced to someone who does not understand its provisions, or to some old promoter whose avocation of building railroads for purely speculative purposes—intending to reap profits out of bonuses and questionable construction contracts—is practically destroyed.

Mr. E. L. Corthell asks:

"Would it not be more reasonable to obtain the contract prices paid by the company in constructing its railroads, and the actual cost from its books when doing the work by its own day labor?"

Assuredly, such a practice would be advisable, and, as a matter of fact, such is the practice actually carried out by the Commission whenever it is possible to obtain the information. When valuing new properties, the Commission requires that all original contracts, for construction of every character, be submitted for its inspection, together with the company's estimates of the actual quantities of all kinds of construction. The estimates of value made up from these are intended to be liberal, and, if anything, more than cover the actual cost. All the original right-of-way deeds, franchises, etc., are filed with the Commission for examination, and a liberal allowance is made for the value of right-of-way, depot and terminal grounds, in accordance with what appears to be the actual market value of the property, including a certain percentage, in addition, for damages that may or may not have been paid to abutting property owners. No deduction is made for the value of property which may have been partially or wholly donated.

The Commission has the figures of actual cost in every case where they can be obtained. In the case of old railroads, existing at the time of the passage of the law, except in a few instances, the construction records, notes and sometimes even all the original maps and profiles, had been destroyed, mislaid or carried out of the State. A number of the lines had been built by construction companies for such stock and bonds as may have been issued, and no records or estimates of cost were ever turned over to the railroad companies.

Mr. Thompson. It should be borne in mind at all times that the Commission was directed by the law to make an "estimate of value," and not an "estimate of cost." Of course, it was the intention of the law that the "estimate of value" should be, to some extent, based on the actual cost, where such cost was reasonable to the company, but the actual cost cannot be adopted as the basis for value in every case. As a physical proposition, a yard of earth has the same value in one railroad as in another in contiguous territory, although the actual cost in one case may have been 10 cents, while in the other it may have been 16 cents, per cubic yard. If 16 cents per cubic yard be considered by the Commission as the value of earth for one railroad, to be consistent, it must also value earth in the other at 16 cents per cubic yard. The fact that one company was able to let its earthwork by contract for less than the other company does not make its earthwork less valuable in the eyes of the Commission.

The writer, certainly, is not one to contend that "seasoning" of the roadbed of a railroad does not in a sense add to its physical value. It is valuable in many ways, *viz.*, the maintenance charges per mile are less, the danger of accidents is decreased, the wear and tear on rolling stock is less, etc. But the question to be decided by the Commission, when establishing its methods of valuation, was whether or not such value was mortgageable, and, if so, how could its value be ascertained. The expense of "seasoning" is properly charged, through roadbed account, to maintenance, and does not appear in the "permanent improvement" or "capital" accounts. It involves no additional outlay of capital by the owners of the road, in the sense that other permanent improvements do, and hence is not value that should be mortgaged; that is, interest charges should not be permitted to be collected thereon. In accordance with the decisions of the Federal Courts, the Commission must permit sufficient rates on freight to enable the railroads to earn, in addition to operating and maintenance expenses, a fair rate of interest on the value of the property. Had it recognized that "seasoning of roadbed" was an item which must be valued in determining the amount of stock and bonds which a railroad could issue, it would have been in the position of imposing a double charge on the public on account of such value, *viz.*, the original cost of such "seasoning" and an annual interest on such cost.

Mr. Corthell is decidedly in error when he states that:

"The estimate on which to base the permitted issue of stocks and bonds is made upon the report of an engineer who goes over the road with a profile and estimates his quantities from center heights."

Up to this time, in the history of the Commission, there has never been an instance where stocks and bonds have been issued on this basis. In all cases that have come up the railroads have been able to supply the Commission with the actual quantities of graduation. The only instances when the approximate method was used were in the

valuations of some of the older railroads, already mortgaged, the actual quantities of graduation for which were unobtainable from any source, and it is probable that, when these properties come up hereafter before the Commission, with application for issuance of new mortgages, accurate surveys will have been made, and, if the estimates offered are considered by the Commission to be correct, they will be adopted by it. Mr. Thompson.

Under the Law, Article 4584 c, "to promote public interests and protect private rights, the Commission may correct its report of value of any railroad at any time it may deem proper." The valuations which have been made and published by the Commission are not necessarily, as Mr. Corthell appears to assume, entirely permanent. A new valuation can be ordered whenever the Commission considers that justice demands it, and the Commission is composed of just and reasonable men. The chief function of the Commission in this particular, as is emphasized by the Stock and Bond Law, is to see that the stocks and bonds hereafter issued by a railroad do not exceed a "reasonable value of its property." Whenever application is made by a railroad for authority to issue stocks and bonds, it becomes the duty of the Commission to ascertain the value of the railroad property at that time, and the value of all permanent improvements, equipment, etc., that have been added since any former valuation, must necessarily be included. Although, up to this time, only one case of this kind has come before the Commission for action, it is considered that under the law it has no discretion in the matter, and must revalue a railroad whenever application is made and the occasion demands.

Since most of the railroads of Texas were valued in 1894 and 1895, just after the passage of the Stock and Bond Law, it must not be supposed that the Commission holds that they are worth no more to-day, as a basis for issuance of securities, than at the time of valuation. The average valuation per mile of those railroads which have been valued is, as stated in the paper, $16 120. It is highly probable that, if a revaluation were ordered to be made now by the Commission, on the same basis as used formerly, this average per mile would be increased several thousand dollars on account of the great amount of permanent improvements and equipment that have been added to the railroads during the past ten years. The value of these improvements, and a reasonable amount for appreciation in all real estate, would have to be added to the former valuations.

An important feature of the Texas Railroad Stock and Bond Law, one that should not be overlooked by anyone attempting to criticise it and the methods of railroad valuation it prescribes, is the enhanced value it has given to railroad securities in Texas. A comparison of the market value of the stocks and bonds of the railroads of Texas to-day with those of ten years ago will convince the most skeptical of this

Mr. Thompson.

fact. The writer has in mind one railroad, which was in the hands of a receiver at the time of the passage of the law, and which recently built an extension. The bonds issued by this railroad, to the value of this extension, under the order of the Commission, sold, on the average, for 108, of itself a handsome profit on the capital involved, exclusive of the liberal donations of land and money which the company received from the communities penetrated. These were 5% bonds, and, had 6% bonds been issued, the limit allowed by law, the profit from their sale, probably, would have been greater. Certain Texas railroad securities which were hawked about the brokers' offices of Wall Street and, with difficulty, found sale at 40 and 50, ten years ago, would now find ready purchasers at par or a little less, had they the stamp of approval of the Railroad Commission.

Another significant fact is that only a short time before the Stock and Bond Law became effective about 39% of the railroads in Texas were in the hands of receivers. To-day there is not a mile, of the 11 300 miles in Texas, in the hands of receivers, and, with a few unimportant exceptions, no railroad has been in the hands of receivers since the law went into effect. The fact is that there has been no piece of legislation, in this or any other State of the Union during the past decade, which has been so fruitful of results and beneficent in its action, alike to the railroads and the people.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 975.

FREEZING AS AN AID TO EXCAVATION IN UNSTABLE MATERIAL.*

By James H. Brace, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. George E. Thomas, D. E. Moran, E. L. Abbott, T. Kennard Thomson, Werner Boecklin, H. J. Campbell, A. Gobert, Charles Sooysmith and James H. Brace.

Introduction.

This paper contains a synopsis of the result of a research of the literature on the above subject. The research was the joint work of H. F. Dose, M. Am. Soc. C. E.; Paul A. Seurot, Assoc. M. Am. Soc. C. E., and the writer.

All articles which could be found in the various libraries of New York City were examined and abstracted, and then combined in the following synopsis. The French and German periodicals were read and translated by Messrs. Dose and Seurot, and those in English were abstracted by the writer. Lists giving the names of the various publications from which information has been obtained have been filed in the Library of the Society, where they may be referred to by those who are interested. Acknowledgments cannot be made in each case, but are due to all the sources thus listed.

History.

For many years the miners of Siberia have taken advantage of the extreme low temperature in winter to penetrate beds of quicksand

* Presented at the meeting of February 17th, 1904.

overlying mineral deposits. Natural refrigeration, however, can have only a limited application in regions of severe winters.

As far as the writer has been able to ascertain, the first use of congelation produced by artificial means, as an aid to excavation, was in England, in 1862. In order to sink a well through a bed of quicksand, a coil of pipes, of a diameter larger than that of the lining, was sunk into the quicksand. Brine, cooled by an ether machine, was circulated through the coil until the material was frozen. The well was then excavated without difficulty.

In 1883, F. H. Poetsch made the first application of his process, at the Archibald Mine, in Saxony. Since that time, progress has been mainly along the line of improvements in his original process.

At least two other processes, the Gobert and the Koch, have been brought forward, but the only essential difference, between these systems and that of Poetsch, is in the medium used for transferring the low temperatures from the machine to the freezing pipes.

The Poetsch Process.

Water-tight tubes, usually parallel to each other, are distributed throughout the mass to be frozen, and are known as the freezing tubes. In practice, they have usually varied from about 4 to 10 ins. in diameter. Inside each freezing tube, smaller pipes, known as the circulating tubes, are placed. Each of the latter has an opening near the bottom into the outer pipe. The circulating tubes are all joined together, usually, by a large circular or rectangular pipe, known as the circulating ring. The freezing pipes are capped at the top and joined to a similar ring, called the collector ring. A circuit is formed by connecting the circulating and collector rings to the cold-brine tank of some form of freezing machine. A pump is placed between the refrigerator and the circulating ring. Cold brine is drawn from the bottom of the tank, forced by the pump through the circulating rings, down the circulating pipes, thence up the freezing tubes, where it absorbs a certain amount of heat, and back through the collector ring to the top of the refrigerator. The circulation is maintained in this manner until an ice wall is formed completely surrounding the area to be excavated.

Freezing Tubes.—The first operation, usually, is boring for the freezing tubes. The holes are put down by one of the well-known

PIPE CONNECTIONS.
CHAPIN MINE.

From the *School of Mines Quarterly*,
Vol. XI.

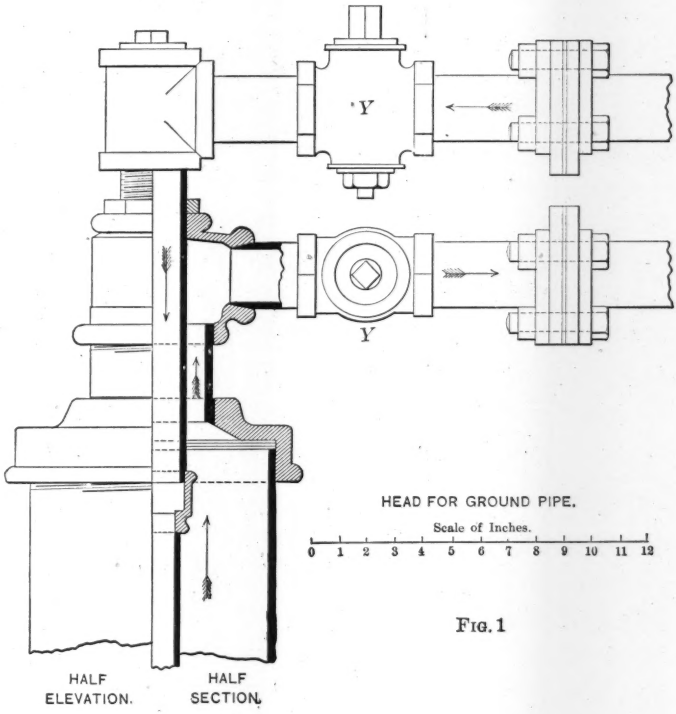


FIG. 1

methods. The one chosen depends largely on the nature of the material. In the earlier applications of the Poetsch process, the casings of the bore-holes were used for the outer or freezing tubes.

Considerable difficulty was found in closing the lower ends of the casings so that the brine could not escape. Also, the joints in the casings were often defective, and leaked, after the severe strains to which they were subjected in being forced down. The escape of the freezing medium, even in small quantities, prevents congelation of the impregnated strata, and thus causes a weak place, if not a leak, in the ice wall. In nearly all the work, of late years, a pipe somewhat smaller than the casing is lowered inside and the casing withdrawn. This pipe is sealed at the lower end, and each joint in turn is tested under heavy water pressure. Another cause of leaks is the contraction of the pipes under the low temperatures.

The partially frozen earth, especially where the shaft is deep, often obtains such a hold on the tube as to render movement impossible. If any further contraction takes place the pipe is ruptured and a leak produced. As a remedy for this, in at least two shafts sunk recently, slip joints have been placed at about mid-depth on the pipes.

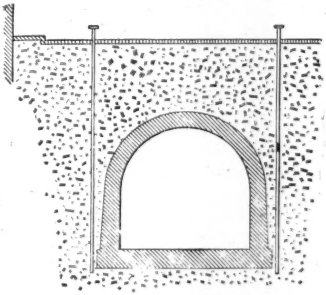
In the first sinkings, 8 or 10-in. pipes were used, but, in late years, the diameter has usually been from 4 to 6 ins.

Circulating Pipes.—The circulating pipes have commonly been from 1 to 1½ ins. in diameter. They are either supported by the fittings at the top, or bear on the bottom of the freezing tubes. In the first case, their lower ends are left open for the passage of the descending freezing liquid. In the latter case, slots are provided in the sides near the bottom for the same purpose. The fittings at the top of both the circulating and freezing pipes are well shown in Fig. 1.

Arrangement of the Freezing Tubes.—In the first shafts built by this process, tubes were distributed throughout the area to be excavated, as well as on a ring outside. Later, only one tube was put down inside the limits of the sinking, and that usually in the center. In some cases this tube was insulated so as to freeze only the bottom of the shaft. In the latest cases, the central tube has been omitted altogether, at least for freezing purposes. It is a common practice to sink a well, of a greater diameter than that of the ring of pipes, down to the ground-water level, and then start the borings from the bottom of the well. In many cases, particularly where the shafts had been

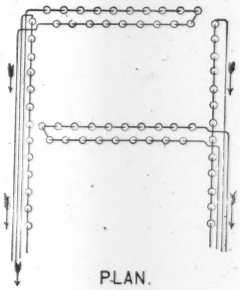
GOBERT PROCESS.

From *Transactions* of the Institution of Mining Engineers, 1902.



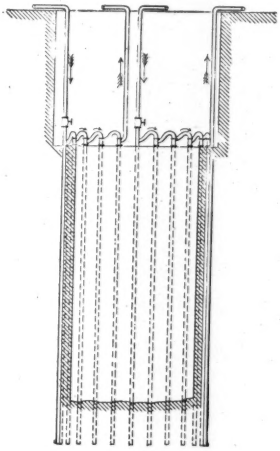
SECTION.

FIG. 2



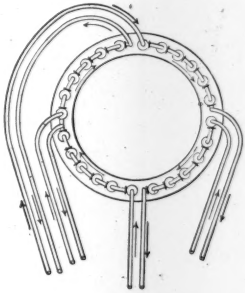
PLAN.

FIG. 3



SECTION.

FIG. 4



PLAN.

FIG. 5

started by other methods, regular arrangements of the tubes were not practicable.

In the case of the Archibald shaft, and in the second attempt at the Jessenitz shaft, all the pipes were placed inside the area to be excavated. It was found that when the pipes were uncovered they would not long maintain the ice wall. This was especially true at the Jessenitz shaft.

Circulating and Collector Rings.—The circulating and collector rings, usually, are made circular or rectangular, according to the arrangement of the outside ring of pipes. The rings are generally placed near the top of the freezing tubes, but experience has shown that the tubes should be extended above the ground-water level, so that all flow into the shaft can be stopped. Valves are provided, on the connection between the rings and tubes, in order that any tube may be cut out in case of leakage. These valves also afford a means of regulation if the freezing action in one tube takes place more rapidly than in its neighbors. It does not appear that they have been used much for this purpose.

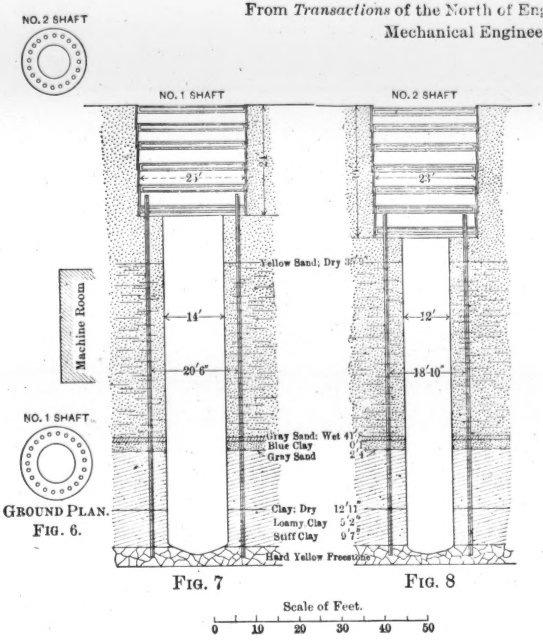
In some cases, as at the Auboué shaft, trouble has been caused by individual tubes becoming choked and stopping the circulation. The stoppage may be caused by scales of rust falling to the bottom of the larger tube and closing the end of the inner tube, or by some foreign substance which gets into the tube through the brine. When the temperature of the air around the tops of the pipes is high, a stoppage of the flow soon becomes apparent by the melting of the frost on the pipes and connections. When the temperature is low, however, as in winter, there is no means of knowing when a stoppage occurs. To remedy this state of affairs, M. Gobert has suggested the arrangement shown in Figs. 4 and 5. By this plan the tubes which receive the cold solution first would have more effect, and the refrigeration would be irregular. To obviate this difficulty, the inventor suggests reversing the current at frequent intervals.

Mr. Cavallier, with the same object in view, proposes to retain the circulating rings, but use separate return pipes from each freezing tube.

Refrigerating Machinery.—Many different types of refrigerating machinery have been used, and any device that will cool the freezing solution down to about —20° or —25° Cent. will answer the purpose.

WASHINGTON SHAFT.

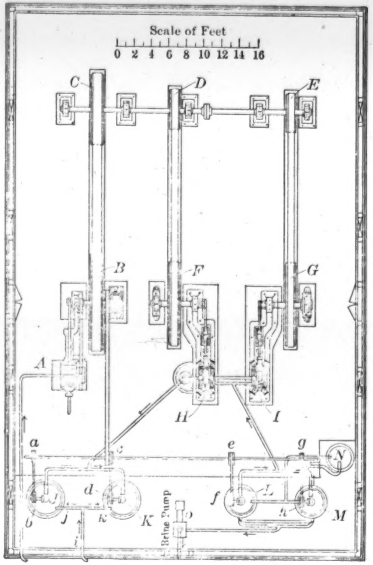
From *Transactions* of the North of England Institute of Mining and Mechanical Engineers, Vol. LII.



GROUND PLAN.
FIG. 6.

FIG. 7

FIG. 8



PLAN.
FIG. 9

It may be interesting to describe briefly the plant which was used at the Washington Colliery, in County Durham, England, shown in plan in Fig. 9. The plant consisted of the driving engine, *A*, the compressors, *H* and *I*, the condensers, *J* and *K*, the refrigerator, *L* and *M*, and the circulating pump. The latter, of course, was a part of the circulating system, rather than of the refrigerating plant. Anhydrous ammonia, NH_3, was compressed to a pressure of 150 lbs. per square inch in the cylinders, *H* and *I*. From the compressors it passed to the condensers, *J* and *K*. These consisted of tanks containing coils of 1-in. pipe, about 1 600 ft. long. Cold water was circulated through the tanks, and was kept in constant motion by the paddles, *p* and *d*. The gas

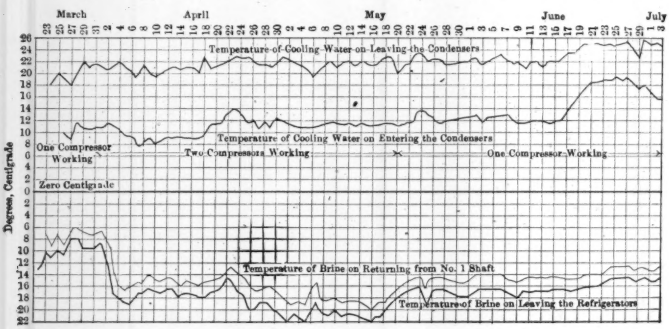


FIG. 10.

from the compressors, in passing through the coils, gave up a considerable portion of its heat to the cooling water, and in doing so changed from a gas to a liquid. About 4 000 galls. of water per hour were required for this purpose, and its temperature was raised about 10° Centigrade.

The refrigerators were similar to the condensers, except that the liquid outside the coils was the freezing solution, and the coils were 2 000 ft. long. The liquid ammonia from the condensers was admitted to the coils of the refrigerators by a valve, which reduced its pressure to 15 lbs. per square inch. By this change of pressure the ammonia again assumed its gaseous form, but in doing so it had to absorb from the surrounding freezing solution an amount of heat corresponding to

PROGRESS OF FREEZING, STRENGTH OF FROZEN MATERIAL, ETC.
From *Colliery Guardian*, Vol. 71.
TO ACCOMPANY DESCRIPTION OF POETSCH PROCESS.

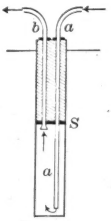


FIG. 11

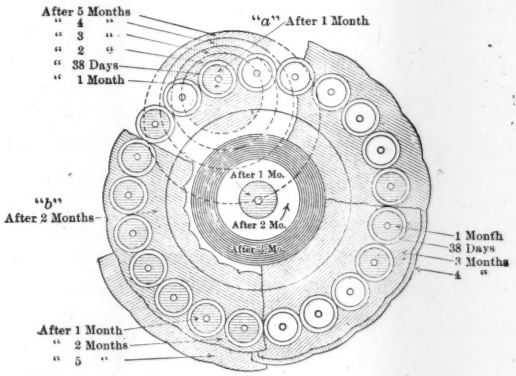


FIG. 12

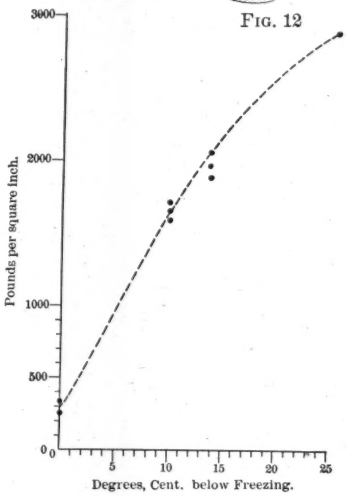


FIG. 13

its latent heat of vaporization. From the refrigerator the gas was returned to the compressor, where it was again compressed and started on a second circuit.

The cold brine was drawn from the bottom of the refrigerator by a pump and forced through the freezing system back to the top of the refrigerator. The brine was circulated at the rate of 144 galls. per minute. The temperatures of the brine, on leaving and returning to the cooling tank during the different stages of the process, are shown in Fig. 10.

The freezing plant is usually located as near the top of the shaft as practicable, in order to avoid loss of efficiency in transmitting the freezing solution. There seems to be some difference of opinion as to the amount of this loss. Mr. Moran states that the pipes above ground at the Chapin Mine were not protected in any way, but "immediately after starting congelation, condensation of moisture coated them with a snow-like ice that served as a cheap and effective lagging." On the other hand, Saclier and Waymel state that the loss at the Vicq Pits, where one plant was used for simultaneous work at two shafts, 121 ft. apart, was 20% at the surface. Arrangements must be made to supply the machine with a considerable quantity of cool water, about 4 000 galls. per hour, and to dispose of it after it has passed through the condensers. If such a supply is not available, means must be provided for cooling the condensing water.

Experience has shown that the freezing plant should consist of at least two complete units, as a total stoppage of congelation may be followed by serious results.

Congelation.—The progress of congelation in uniform material is well illustrated in Fig. 12. During the first stage, cylinders of ice form around the individual pipes, and increase uniformly in all directions until adjacent cylinders touch and form a closed wall. In the second stage, after the wall is formed, each cylinder tends to increase in the manner shown by the dotted lines around the tube, *a*. The cylinders, however, interfere with each other, and produce the fluted annular ring, *b*. This ring increases in thickness much more rapidly inside the ring of the pipes than on the outside. This is explained by the facts that the loss by conduction is much less toward the center, and that the area affected by each tube is greater on the outside. Where deep shafts penetrate impervious beds overlying heavily water-

bearing strata, some apprehension has been felt for the safety of the ice wall. This condition is shown in Fig. 14, illustrating the conditions at Harchies. After the impermeable clay, which congeals much more rapidly than the greensand, has completely frozen inside the circle of pipes, there will be no escape for the water confined in the space, *A*, and expanding constantly under the influence of the lowering temperature. To meet this contingency, the central pipe shown in Fig. 14 is provided. This pipe is described more fully in the account of the Harchies shaft.

The difficulties of congelation are increased by running water, or by the presence of salt water or other substances which do not freeze readily.

The strength of the frozen material varies greatly with the nature of the material itself, with the quantity of water present, and with the temperature.

HARCHIES SHAFT.

From *Engineering and Mining Journal*, Vol. LXVII.

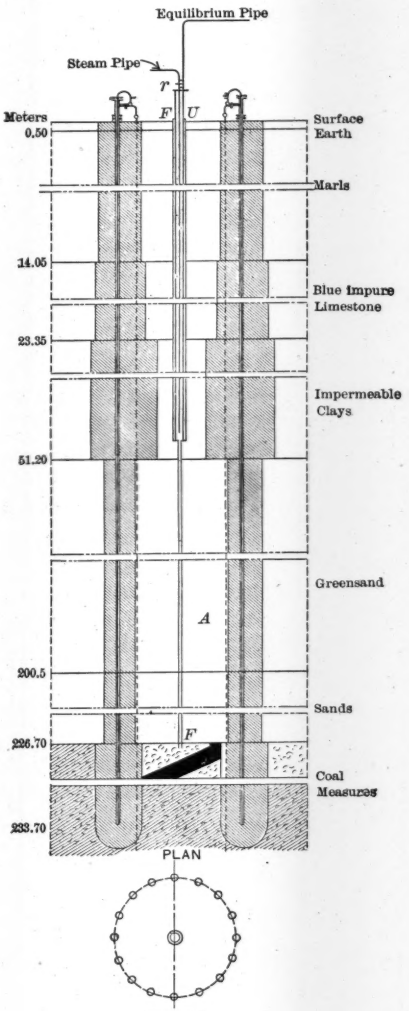


FIG. 14.

The results of some tensile tests, made by Mr. Alby, in France, on saturated sand, are shown graphically in Fig. 13. Tests made at the Chapin Mine (temperature not recorded) gave average results as follows:

Tensile strength, 431 lbs. per square inch.

Compressive strength, 575 lbs. per square inch.

Experience has shown that it is desirable to preserve an uncongealed core in the center of the shaft, as excavation is facilitated greatly thereby. Care must be taken, however, that the annular ice wall is carried well below the water-bearing material, or else the lower end of the cylinder must be frozen solid. The latter can be accomplished by the insulated central tube mentioned previously and illustrated in Fig. 11.

Form and Required Thickness of Ice Wall.—Theoretical discussions of the form and required thickness of the ice wall are given at considerable length in the *Annales des Mines*,* and in the *Bulletin de la Société de l'Industrie Minérale.*†

Lack of knowledge, as to the pressures to be sustained, the strength of the frozen material, the conductivity of the various strata, etc., renders the results of analytical study of doubtful value.

Saclier and Waymel, Engineers of the Vicq shafts, state that the freezing pipes should be placed on a circumference of 2 ft. greater radius than that of the excavation. This statement appears to represent practice fairly. Excavation is seldom started until the ice wall is thick enough to encroach on the area to be excavated, and, accordingly, the thickness is rarely less than 3 ft.

It was formerly believed that the form of the wall was that of a truncated cone with its greater diameter at the base.

Observations have shown that the frozen mass may assume any of the following forms: A truncated cone with the largest diameter at the bottom; a truncated cone with the largest diameter at the top; a cylinder; a series of cylinders of varying diameter; a barrel shape.

The base may be either flat, concave or convex.

Time Required for Congelation.—This varies greatly with the nature of the material and the thickness of the ice wall. For twenty-one sinkings, of which the data are available, the minimum initial freezing

* For the year 1885, p. 111; 1887, p. 56; 1900, pp. 379-486.
† For the year 1895, p. 273.

period was 15 days, the maximum 365 days, and the average 94 days. The initial freezing period covers the time from starting the ice machine to beginning excavation. Congelation is commonly continued until the shaft is completed. During the latter period, however, the full capacity of the machine is seldom required.

Excavation.—Ordinarily, the frozen material is about as difficult to excavate as soft sandstone. Formerly, it was excavated entirely by the use of picks, wedges and chisels. More recently, compressed powder, dynamite and gelignite have been used in several instances.

Apparently, the use of explosives has not caused any damage to the ice wall, but some breaks in the freezing tubes have been attributed thereto. In drilling the frozen material, brine must often be used, as water freezes and holds the drills fast in the holes.

Lining.—Timber, masonry and iron are commonly used for the lining. Timber and iron linings are usually backed with concrete. In most cases it has been found necessary to add some substance to the mortar to prevent injury from freezing.

Mr. M. F. Schmidt* states that when iron lining is used, in order to avoid undue contraction and expansion due to extreme changes of temperature, it is well to protect the lining by placing between it and the ground some non-conductor, such as wood, straw or ground charcoal. As to masonry, it is desirable to use a mortar mixed with salt water and pitch, and, moreover, to insulate the lining by a wooden sheathing. From the information at hand it does not appear that the insulation has been used in many cases.

Subway Work or Tunneling.—Figs. 2 and 3 illustrate the method suggested by Gobert and others for applying the freezing process to the construction of tunnels. Thus far, however, no actual trial has been made.

Gobert Process.

This process was invented by A. Gobert. The details have been worked out quite carefully, and a great deal has been written about the system, but, as far as the writer can learn, it has never been applied in practice.

The essential difference between the Gobert and Poetsch systems is in the medium used for cooling the ground. Gobert dispenses

* *Bulletin de la Société de l'Industrie Minérale*, 1895, p. 273.

with the refrigerator and cooling brine, and allows the liquefied ammonia to expand in the freezing tubes themselves. Some modifications of the fittings are required, principally in the circulating tubes. If the liquid ammonia were allowed to pass directly to the bottom of the freezing tubes it would collect there, and its evaporation would be very slow. Spiral tubes, perforated at frequent intervals, are proposed, to allow the escape of the liquid for the full height of the material to be frozen. Some of the fittings invented by Mr. Gobert are shown in Figs. 15, 16, 17 and 19.

One of the main advantages claimed for this system is that there is no tendency for the freezing medium to escape into the ground to be frozen.

In the Poetsch system, the pressure at any point in the freezing pipes must be equal to the weight of the column of brine above that point. This is always greater than the pressure on the outside of the tubes. In the Gobert system, there is only the pressure of the expanded gas, which is always less than the outside pressure. Accordingly, if breaks occur in the freezing pipes, the leak will be inward instead of outward.

Koch Process.

The writer has been unable to find any very complete description of this process. It involves the direct use of the liquefied gas in the freezing tubes. The gas used may be either carbonic acid, ammonia or a mixture of dioxide of sulphur.

The freezing tubes are divided into various compartments to facilitate expansion. It is claimed: First, that a uniformly distributed temperature of —50° or —60° Cent. is obtained; second, that this low temperature is produced as soon as the machine is started, and, therefore, the freezing tubes contract to the full extent before they are frozen in place, there would be no tendency to rupture the pipes, as in the Poetsch process, where the reduction of the temperature is gradual; third, that the time required for congelation is reduced one-half. The freezing machine and connections are shown in Fig. 20.

From the information at hand, it appears that this process only differs in detail from that of Gobert, but Louis Gebhardt is quoted as

GOBERT PROCESS.

From *Transactions* of the Federated Institution of Mining Engineers, Vol. XI.

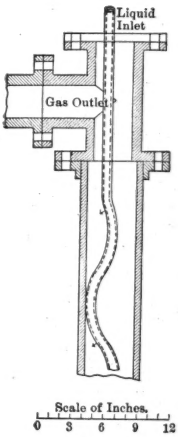


FIG. 15.

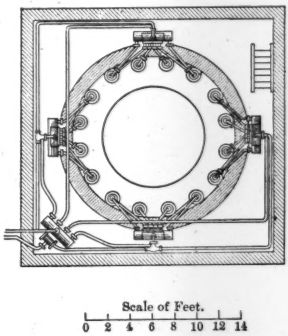


FIG. 19.

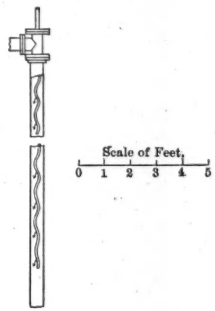


FIG. 16.

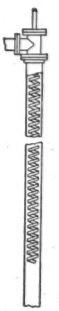


FIG. 17.

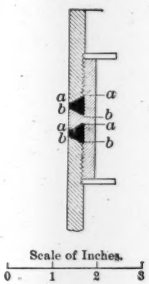


FIG. 18.

saying: "The Koch process will solve all difficulties met in actual practice." No accounts have been found of any actual application of the system in sinking a shaft or driving a tunnel.

Freezing by Cold Air.

The only direct application of cold air that has been found was at Stockholm, and that is described under the head of the Stockholm Tunnel. This is also the only case found where the freezing process has been applied to tunnel work.

Application of Freezing Processes.

Archibald Shaft.—This shaft belongs to the Douglas Company, and is located at Schneidlingen, Saxony. This was the first practical application of the Poetsch process. The work was done during the year 1883. A shaft, 15.4 x 10.3 ft., was sunk to a depth of 111.5 ft. through solid and fairly compact measures by pumping. At this depth very watery quicksand was encountered. A bore-hole was put down, through the quicksand and underlying measures, into a previously existing heading of the mine. The object of the bore-hole was to drain the quicksand, and a considerable flow of water was set up through the channel. Sinking was then resumed by means of sheet-piles, but, after an advance of 4.3 ft., the work had to be abandoned, and it was turned over to Mr. Poetsch. The material to be congealed consisted of 18 ft. of quicksand. Twenty-three wrought-iron tubes were put down through the quicksand and 1 ft. 7 ins. into the lignite. Owing to the difficulty of enlarging the shaft, all the freezing pipes were placed inside the area to be excavated; ten of them were placed close to the sides of the shaft. Their arrangement is shown in Fig. 21. The pipes were 8 ins. in diameter, and were closed at the bottom by inserting wooden plugs. The plugs, in turn, were covered with 2 ins. of cement and 2 ins. of plaster and clay. The inner or circulating tubes were about 2 ins. in diameter. The distributing and collecting rings were placed at the level of the top of the freezing tubes, or 111.5 ft. below the surface.

A Carré-Kropff machine, capable of producing half a ton of ice per hour, was used for freezing. The freezing solution was chloride of calcium. To observe the progress of congelation, twenty small pipes,

KOCH PROCESS.

From *Gluckauf*, 1895.

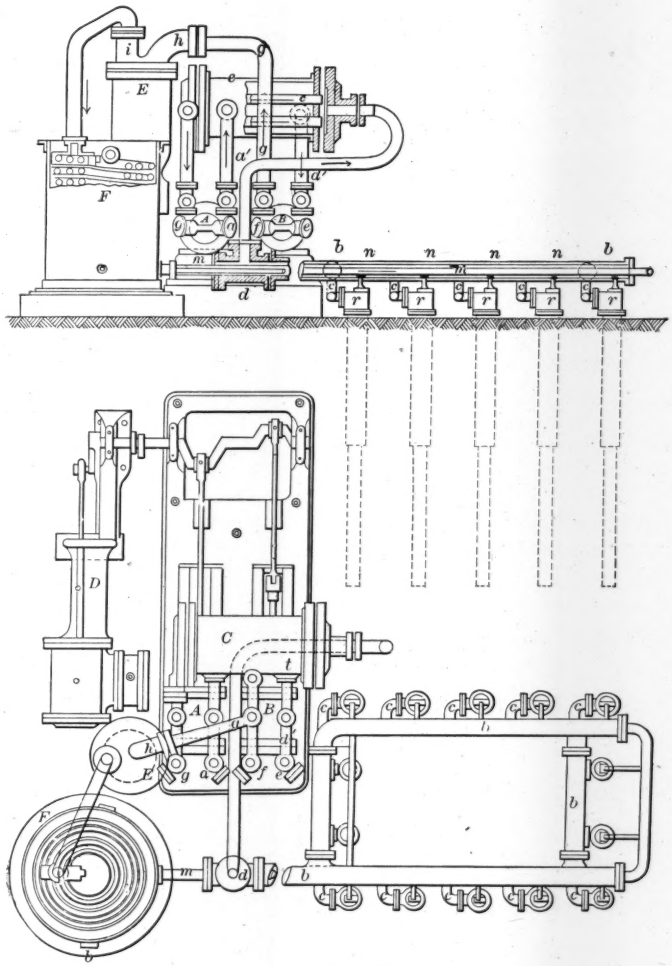


FIG. 20

20 ins. in length, were sunk into the quicksand and filled with a solution of chloride of calcium. Inside these pipes thermometers were placed.

Table No. 1 shows the temperature at various points in the shaft at the beginning of the freezing process and at the end of the initial freezing period.

TABLE No. 1.

	July 8th.	July 31st.
Temperature of air at bottom of shaft.....	+ 53.6° Fahr.	21.2° Fahr.
" " ground, east side.....	51.8 "	— 2.2 "
" " " south "	51.8 "	— 0.5 "
" " " west "	51.8 "	— 2.2 "
" " " north "	51.8 "	— 0.4 "

The temperatures of the ground at the sides were taken as near as possible to the freezing tubes. Owing to the fact that the upper ends of the freezing pipes were 4.3 ft. above the quicksand, there was considerable loss by radiation. The stream flowing through the center of the shaft also had the effect of warming up the freezing solution and of retarding the progress of congelation. Fifteen days after the freezing machine was started, the ground over the entire area of the shaft, and about 30 ins. outside, was frozen solid.

A small shaft was first excavated to the coal. This shaft was timbered, temporarily, every 3 ft. Coal was reached on September 30th. Enlargement of the shaft to full size was then started at the bottom, thus uncovering all the freezing tubes. It was found that radiation from the pipes could not be depended upon for maintaining the ice wall, as the exposed pipes were coated with hoar frost rapidly. Excavation had to be suspended until the wall had attained such a thickness that it would not be in danger of thawing out during the remaining operations. Excavation was carried on by means of picks, no powder being used. The shaft was perfectly dry, and the soil was as hard as sandstone. It was worked by means of picks, wedges and mauls. The temperature at the bottom of the shaft, while the sinking operations were in progress, varied from 0.5° to 1° Centigrade. The men did not suffer any bad effects from the cold. The permanent lining of the shaft consisted of timber cribs. It is stated that perfectly dry timber must be used, as frost is likely to shatter green or wet timber.

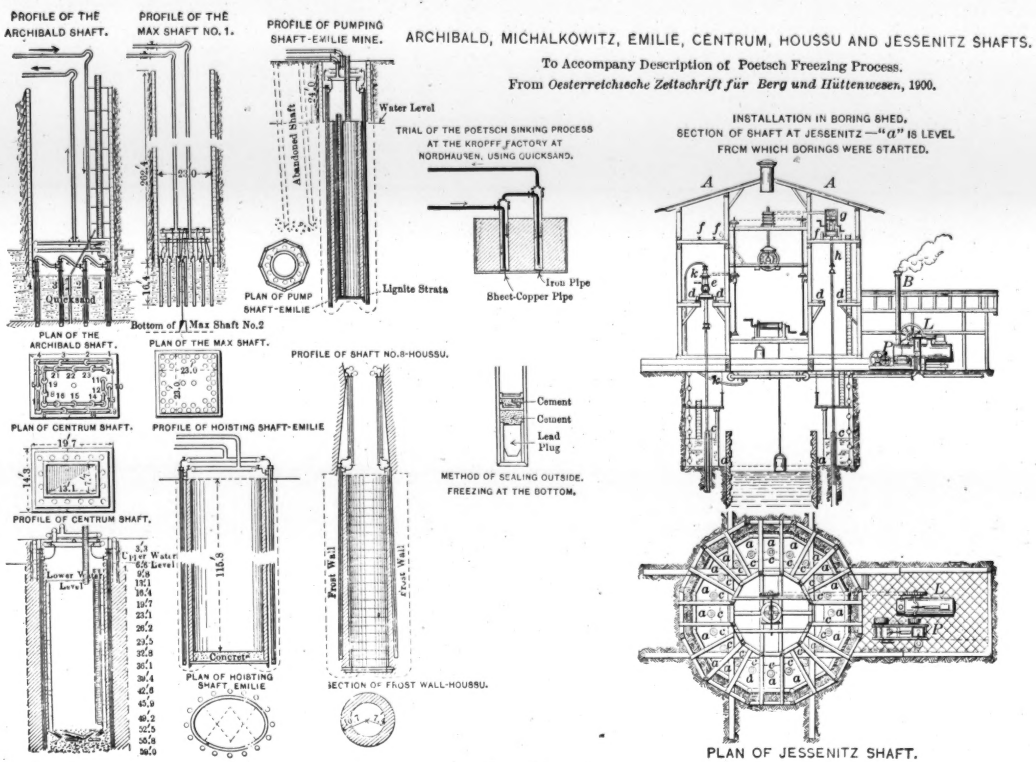

ARCHIBALD, MICHALKOWITZ, EMILIE, CENTRUM, HOUSSU AND JESSENITZ SHAFTS.
To Accompany Description of Poetsch Freezing Process.
From Oesterreichische Zeitschrift für Berg und Hüttenwesen, 1900.
PROFILE OF THE ARCHIBALD SHAFT.
PROFILE OF THE MAX SHAFT NO. 1.
PROFILE OF PUMPING SHAFT-EMILIE MINE.
Water Level
Abandoned Shaft
Lignite Strata
PLAN OF PUMP SHAFT-EMILIE
TRIAL OF THE POETSCH SINKING PROCESS AT THE KROPFF FACTORY AT NORDHAUSEN, USING QUICKSAND.
Iron Pipe
Sheet-Copper Pipe
INSTALLATION IN BORING SHED.
SECTION OF SHAFT AT JESSENITZ—"a" IS LEVEL FROM WHICH BORINGS WERE STARTED.
Quicksand
Bottom of Max Shaft No.2
PLAN OF THE ARCHIBALD SHAFT.
PLAN OF THE MAX SHAFT.
PROFILE OF SHAFT NO.8-HOUSSU.
PLAN OF CENTRUM SHAFT.
PROFILE OF HOISTING SHAFT-EMILIE
Cement
Cement
Lead Plug
METHOD OF SEALING OUTSIDE, FREEZING AT THE BOTTOM.
PROFILE OF CENTRUM SHAFT.
Upper Water Level
Lower Water Level
Concrete
Frost Wall
Frost Wall
PLAN OF HOISTING SHAFT, EMILIE
SECTION OF FROST WALL-HOUSSU.
PLAN OF JESSENITZ SHAFT.

FIG. 21.

The following table of costs of the different operations is attributed to the manager, Herr Frohberg:

Placing pipes	$978.17
Sinking (131.2 ft.)	583.98
Lining	851.64
Congelation (100 days)	900.30
Interest on capital and sinking fund	3 649.88
Carriage of plant	973.30
Erection of plant	1 751.94
Superintendence and traveling expenses	1 800.60
Total	$11 489.81

Michalkowitz Colliery.—The Max workings are located at Laura Hütte, Province of Silesia, Germany. Sinking by freezing was tried at this shaft during the year 1884. An attempt was made to sink two shafts, 164 ft. apart. The first was excavated to a depth of 213.3 ft. by ordinary means. At that depth quicksand was encountered, but work was continued by the use of sheet-piles until the bottom of the pit was 246.1 ft. below the surface. As the water pumped amounted to 205 cu. ft. per minute, work had to be stopped, and the bottom of the shaft was closed with a bulkhead, 3.3 ft. thick.

The nature of the material is shown by the following:

Alluvial sand	45.9 ft.	Hard shell lime	62.3 ft.
Clay (little water)	62.3 "	Variegated sandstone	91.9 "
		Total	262.4 "

At this depth coal was reached.

The shaft was square in plan, 22 ft. 11 ins. on a side.

Forty-two freezing pipes, of 8 ins. inside diameter, were put down in the bottom of the shaft. The manner of placing and connecting the tubes is shown in Fig. 21. The tubes were given a slight inclination outward, and, at their lower ends, were provided with a shoulder for receiving a lead plug. On attempting to place the lead plugs, it was found that the pipes were not all of the required diameter, and, moreover, some of them had assumed an elliptical form under the pressure of driving. Under these circumstances, there was a great deal of difficulty in closing the ends. After congelation was started,

it was found that eleven pipes leaked. Smaller pipes, closed at their lower ends, were placed inside the defective tubes, and operations were resumed.

A Carré-Kropff machine, capable of making half a ton of ice per hour, was used for congelation. After the measures had been cooled to +2° Cent., the machine broke down, and freezing was stopped for a week. During this delay the ground resumed its original temperature. Meanwhile, the owners of the colliery had proceeded with the second shaft, without encountering as much difficulty as in the first.

When they reached a level lower than the bottom of Shaft No. 1, the water was drawn from the first shaft, and there was no longer any need for the freezing process. Soon after this the time limit, under which Mr. Poetsch had contracted to complete the freezing, expired. The temperature of the measures was still at + 1° Cent., and the attempt at congelation was discontinued. The failure at this point is attributed to delays due to the timber in the shaft and the accidents to the plant itself. It is claimed, also, that the pumping from the second shaft retarded the freezing.

Emilie Colliery, Pumping Shaft.—This colliery is located near Finsterwald, Brandenburg, Prussia.

The sinking by the freezing process was commenced in 1884 and completed in 1885. An attempt was made to sink both the pumping and the winding shafts by ordinary means, but both failed and they were filled up. The freezing process was then adopted, but new sites were chosen.

The material penetrated was as follows:

Yellow sand.....	3.3 ft.	Rough gray sand, very aquiferous.....	19.7 ft.
Fine white sand.....	6.6 "	Coarse gray sand.....	21.3 "
Coarse gray sand	11.5 "	Fine gray sand.....	11.5 "
Coarse gravel.....	3.9 "	Fine brown sand.....	13.6 "
Dry, rough sand	7.5 "	Total.....	125.2 ft.
Damp, rough sand.....	26.3 "	Lignite.....	14.8 "

The strata are all very aquiferous. The natural ground-water level was at a depth of 29.5 ft.

The shaft has a clear diameter of 8.8 ft.

A square pit, 16.5 ft. on a side, was sunk to a depth of 26 ft., and from the bottom twelve borings were made on the circumference of a circle having a diameter of about 14 ft. The bore-holes were inclined slightly outward. The linings of the bore-holes, plugged at their lower ends with lead and concrete separated by iron washers with rubber joints, served as the freezing tubes. The depth of the pipes varied from 125.5 to 128 ft., and the penetration into the lignite from 0.7 to 3.1 ft. The relative positions of the old and new shafts, as well as the arrangement of the freezing tubes, are shown in Fig. 21. It was found that Hole No. 2 varied too much from the perpendicular, and two more holes were put down to take its place. This work was not completed until congelation had been begun.

A Carré-Kropff machine, having a capacity of half a ton of ice per hour, was used. The freezing solution was calcium chloride. The machine was running regularly by June 25th, and on August 8th the tower was formed, except near Holes Nos. 1 and 2. Sinking was started before this gap was closed, but water came in, stopping the excavation. Attempts to continue by pumping weakened the ice wall, but, on September 8th, it was sufficiently strong to allow the resumption of excavation.

The shaft was lined with timber temporarily, and, on October 31st, the coal seam was reached. It was found that the center was not frozen, the congealed matter having taken the form of a bottle. Before the temporary lining could be completed, water burst in, and the bottom was filled with gravel. By continuing congelation the leak was stopped. Timber cribs were inserted next, near the top of the lignite, forming the base for the permanent masonry lining. On November 20th, when the height of the permanent lining was 16.4 ft., the machine was stopped for four days, when the ice wall gave way and sandy water rose between the masonry and the sides of the pit. This leak was checked by starting the machine, but, on December 26th, a fresh inburst occurred. It was found necessary to fill the shaft with ice and snow, after which the walls were congealed thoroughly in a short time. The lining was completed about April 1st, 1885. Most of the trouble with the ice wall is attributed to starting the excavation before the congelation had been completed, and to the fact that the pipes were not sunk deep enough into the coal.

The cost is summarized as follows:

Allowance for interest and cost on plant, 25 per cent. of $14 610	$3 652.50
Cost of carriage of plant	974.00
Erection and establishment	4 675.20
Use of freezing machine two hundred and forty days	2 834.34
Sinking and lining	1 850.60
Superintendence, etc.	1 860.34
Total	$15 846.98
Cost per foot	$126.80

Emilie Colliery, Winding Shaft.—This shaft was located 147.6 ft. from the pumping shaft. It was built during the year 1885.

A rectangular shaft was sunk by ordinary means to a depth of 32.8 ft., or about 3 ft. below the water level. Below this depth, the shaft is an ellipse having a major axis of 13 ft. 2 ins., and a minor axis of 10 ft. 1 in. A plan and vertical section are shown in Fig. 21. Sixteen refrigerating pipes, about 1 m. apart, were put down surrounding the ellipse. The total depth of the holes varied from 117.3 to 121.4 ft., and the penetration in the lignite was from 1.0 ft. to 3.8 ft. Sinking was begun on June 11th, 1885, and had reached a depth of 94.3 ft. on August 15th, when it was found that some of the pipes were not tight at the bottom. Excavation had to be suspended until October 9th, to enable fresh plugs to be put in. Near the bottom of the shaft the ice wall gave way on several occasions, and the pit was filled partially with sand. The lignite was reached on November 5th, and shortly afterward the masonry lining was started. The mortar in the lining was prevented from freezing by steam led through a copper pipe. The lining was completed on December 12th, 1885. Borings showed the thickness of the ice wall to be 2 ft. or more, down to a depth of 98 ft. At that depth it decreased to 10 ins. quite suddenly on one side, and water and sand burst into the shaft. The decrease in thickness was attributed to leaking pipes. The cost per foot averaged about $101.00.

Centrum Colliery.—This colliery is located at Köningswursterhausen, Brandenburg, Prussia, and is owned by Dr. W. Siemens.

Most of the work was done during the year 1884. An attempt was

made to sink the shaft by pumping, but this method had to be abandoned at a depth of 52.5 ft. The Poetsch process was then adopted.

The material through which the shaft was sunk is as follows:

Yellow sandy clay	14.7 ft.	Coarse sand	2.1 ft.
Gray sandy clay	6.6 "	Sandy clay	13.9 "
Gray sand	0.5 "	White quartzose sand	54.0 "
Sandy clay	13.0 "	Total	104.8 ft.

The ground-water was at a depth of 14.8 ft.

For a depth of 19.7 ft. the old excavation was enlarged to a rectangular section, 14 ft. 2 ins. x 19 ft. 8 ins. Below this depth the shaft was also a rectangle, but only 13 ft. 1 in. x 7 ft. 7 ins. Fig. 21 shows a plan and vertical section.

At the bottom of the enlarged shaft, sixteen freezing pipes, four on a side, were put down parallel to the sides of the small shaft and outside its area. The freezing pipes were 7 ins., outside diameter, and $\frac{5}{16}$ in. thick, and their bottoms were 106.6 ft. below the surface of the ground. The lower ends were closed with lead plugs. The 7-in. pipes ended a short distance above the bottom of the enlarged shaft, and were connected with the collector ring, which was above the surface of the ground, by smaller bent pipes.

Congelation was started on April 14th, and sinking on June 8th. During the greater part of this time, the water stood in the shaft at its natural level.

The ground in the middle of the shaft was thawed by steam, and by this means was formed a shaft the sides of which were attacked by picks. When the excavation had reached a depth of 52.5 ft., a break in the ice machine caused a suspension of congelation for a month.

The surface water flowed down over the frozen wall and raised the temperature to +10° Centigrade. Accordingly, the work of sinking was stopped and the shaft filled with water. Two weeks after the machine was started, excavation was again resumed, and the coal was reached on October 11th. The daily progress was about 2.8 ft.

The timber lining was completed, except the last crib at the bottom, when a bore-hole was put down into the lignite which was found to be frozen to a depth of 110.9 ft. below the surface of the ground. On the following day, water burst into the shaft, through the bore-

TUNNEL FOR FOOT PASSENGERS, STOCKHOLM.

From *The Engineer*, London.

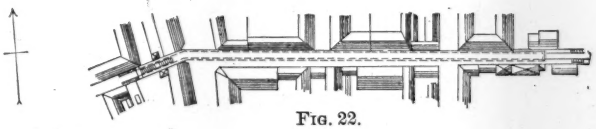


FIG. 22.

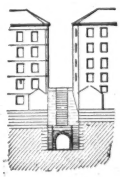


FIG. 23.

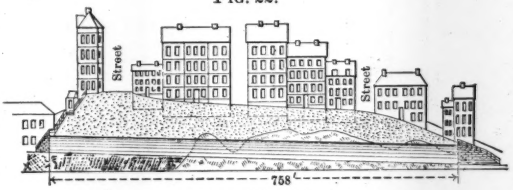
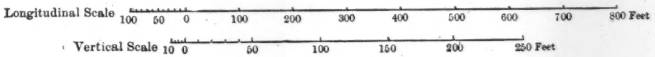


FIG. 24.



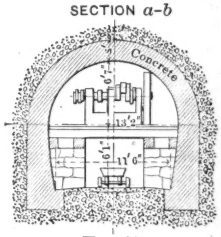


FIG. 25.

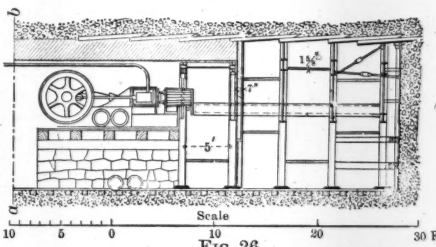


FIG. 26.

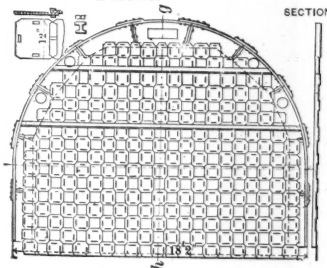


FIG. 27.

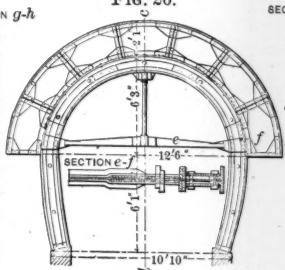
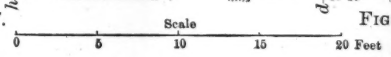


FIG. 28.



hole, and enlarged the channel rapidly. The shaft was pumped out on several occasions, but the lining was not completed. A second accident to the ice machine added to the difficulties, and, finally, a plug at the bottom of one of the freezing pipes blew out, allowing the liquid to escape. The latter accident made the completion of the lining impossible. The shaft, however, was used for pumping purposes for a couple of years, or until the mine was abandoned for reasons not connected with this shaft.

The cost is stated by Mr. Poetsch to have been $162.00 per foot.

The failure to complete this shaft satisfactorily is said to have been due to not sinking the freezing tubes far enough into the lignite, and to delaying the completion of the lining.

Stockholm Tunnel.—This tunnel is in Stockholm, Sweden, and was built during the years 1884 to 1886, inclusive.

It was designed and built by Captain Lindmark. The object was to afford easy communication for pedestrians between two parts of the city separated by the ridge, a cross-section of which is shown in Fig. 24. The eastern end of the tunnel, being mostly in rock, was constructed in the usual manner, and caused little trouble. At the west end the Rziha system was at first adopted. The iron wall shown in Figs. 26, 27 and 28 was also devised for supporting the face. After an advance of 40 ft. by the above means, a considerable subsidence had taken place, and work had to be discontinued. Captain Lindmark then decided to freeze the earth before proceeding farther.

The tunnel is 12 ft. 8 ins. high and 13 ft. 2 ins. wide. The cross-section is shown in Fig. 25. The length is 758 ft. Fig. 22 is a plan of the work, Fig. 23 is a cross-section, and Fig. 24 is a profile of the street and tunnel. The relation of the rock and earth is shown clearly by the profile. The rock was granite. For the first 120 ft. from the west end, the material consisted of coarse gravel intermixed with large stones and a small quantity of wet clay. On account of the water it contained, this material had no cohesion whatever, and would run through a very small opening. The remaining portion of the tunnel, before entering the rock, penetrated pure sand possessing considerable cohesion.

For freezing the earth, a Lightfoot dry-air machine was used. It was capable of delivering about 25 000 cu. ft. of air per hour. The air left the machine at a temperature of —55° Centigrade. The

machine was erected as close to the heading as practicable, and a freezing chamber was formed by partitioning off the heading. The partition was movable and consisted of a double wall of planking filled with charcoal. After the machine had been discharging cold air into the freezing chamber for sixty hours, it was found that the surrounding material had been frozen to depths varying from 5 ft. at the bottom to 1 ft. near the top. At the crown no freezing took place. The temperature at the bottom was as low as —40° Cent., but that of the roof remained steadily at 0° Centigrade.

Excavation was started at the top, the roof being supported by poling boards driven into the uncongealed material at the crown. This soft material was considered an advantage, as it is said the roof would have had to be supported in any event, and the driving of the poling boards was made much easier thereby. The heading was advanced in lengths of 5 ft., and the face was supported by the iron wall built from the top downward to within about 8 or 9 ft. of the floor. The material was so solid that it was not found necessary to support the lower portion of the face. After the first, the freezing machine was only run from 10 to 12 hours during the night. At the end of that time the temperature was from —20° to —25° Cent., but soon after the workingmen entered it rose to zero. After heavy rains it was necessary to run the machine somewhat longer. When two sections had been completed, the partition was moved forward. The size of the freezing chamber varied from 3 000 to 6 000 cu. ft. A length of about 80 ft. of tunnel was constructed by this means. The material then became sufficiently firm to allow the excavation to proceed without freezing. The rate of progress while congelation was used was about 1 ft. per day.

The lining was built as rapidly as possible after the partition had been moved forward and while the ground was still frozen. The concrete used was composed of 1 part of Portland cement, 2½ parts of sand and 6 parts of granite. The lining did not appear to suffer any injurious effects from the frozen material behind it.

No subsidence took place on the north, but the house on the south settled about 1 in., producing some small cracks.

Houssu Colliery, Pit No. 8.—This colliery belongs to the Société Anonyme des Charbonages de Houssu, and is situated at Haine-Saint Paul, in the Province of Hainaut, Belgium.

The Poetsch process was adopted in 1885, and the work was not completed until near the end of 1887. At first, an attempt was made to sink by pumping, as a bore-hole, some 50 ft. away, did not disclose any unusual difficulties. At a depth of 82 ft. a stream of water, yielding about 7.3 cu. ft. per minute, was encountered, but the pumps were able to handle this quantity. When the bottom of the pit had been carried down to a depth of 196 ft., and, while excavating in hard marl, quicksand and water burst in through the floor. The water rose to a point 82 ft. below the surface, and the lower portion of the excavation was filled with quicksand to a height of 20.3 ft. The thickness of the solid marl through which this inrush took place was determined afterward to have been 4.9 ft. Attempts to take out the water and proceed with the excavation were without result, and, as the best means for saving the work done, the freezing process was adopted. The shaft is circular, and its internal diameter is 13 ft. 1 in. There is some conflict as to the nature of the material which was to be congealed.

The following represents its nature approximately:

Sand which had flowed into the pit	19.7 ft.	Sand, with pyrites	1.6 ft.
Hard marl	4.9 "	Black sand, with lignite and shells	5.8 "
Sand, with pebbles	15.1 "	Black sandy clay, pebbles, pyrites and lignite	1.3 "
Black sand, with traces of clay	1.0 "	Fine-grained black sand	1.3 "
Argillaceous sand	0.9 "	Black earthy and sandy clay, with pyrites and fossil wood	3.3 "
Sandy clay	2.3 "	Black argillaceous sand, with pyrites	2.0 "
Consistent gray plastic clay	1.2 "	Total	61.4 ft.
Black sand	0.5 "		
Green gravel, with shells	0.5 "		

After clearing the water from the pit, its size was increased gradually from the depth of 144.3 ft. to that of 177.2 ft., so that its diameter at the latter depth was 19 ft. 8 ins. From the ledge thus formed, on the circumference of a circle having a diameter of 16.79 ft., eighteen freezing pipes were put down. The bottoms of the tubes were at a depth of 246 ft. below the surface, and, therefore, they were 69 ft. long. The outside or freezing tubes were 6 ins. in diameter, and the inside or circulating tubes 2 ins. The boring for and placing of

tubes occupied five and a half months. The circulating and return rings were placed near the level of the bench, and were connected with the freezing tubes in the usual manner. A single well-lagged pipe led from each of the rings to the cooling apparatus, thus completing the circuit. The arrangement of the tubes is shown in vertical section in Fig. 21.

Congelation was started on December 12th, 1885, with a Carré-Kropff machine capable of producing about 900 lbs. of ice per hour. After six months, the temperature of the sand was found to be at 0° Cent., and it was not sufficiently solid to allow the work of excavation to be begun. Mr. Poetsch attributes the inefficiency of the first machine: First, to its distance from the freezing tubes, 177.1 ft.; and, second, to the influence of the warm water said to come from the condenser of a 200-H.-P. engine at another shaft. He estimates that these two sources absorbed at least 50% of the output. The water had been allowed to stand at its natural level, 82 ft. below the surface.

By the advice of Mr. Poetsch, the pit was kept dry by pumping, and the capacity of the freezing plant was doubled, but about five months more elapsed without result. Mr. Poetsch then discovered that one of the freezing pipes was broken, allowing the uncongealable liquid to escape, and, also, that the large quantity of water pumped had a temperature of 11° or 12° Centigrade. The broken tube was repaired and pumping stopped. The freezing of the ground was then completed successfully in four weeks. The process of congelation consumed one year. The freezing liquid was a solution containing about 23% of calcium chloride. During the latter part of the process the temperature of the liquid on leaving the machine was about —25° Cent., and on returning it was about 4° higher. Sinking was carried on by means of picks and wedge needles. A shaft, about 11 ft. square, was put down to the coal first, by gangs of four men working in 6-hour shifts. After excavating a depth of 8 ft., temporary timbering was put in and excavation resumed. The rate of progress was about 1 ft. 7 ins. per day. An attempt was made to thaw the earth by means of steam. Five cylinders of plate iron, 3 ft. $3\frac{3}{8}$ ins. high and 2 ft. 7 ins. in diameter, were placed on the shaft floor. The cylinders were open at the bottom, and steam was admitted at the top. This apparatus was too cumbersome, and, besides, it produced a mud that was very disagreeable to work in, and its use was soon abandoned.

After the completion of the advance shaft, it was enlarged from the bottom up and the shaft lined with cast iron, 1½ ins. thick. The iron lining was backed with concrete. The cement was found to set satisfactorily. Sea salt was added to the mixing water at first, but was found to be unnecessary. The iron lining was carried up to the base of the old work, where it was joined with a masonry lining placed previously.

At the depth of 177 ft. below the surface, work on the lining was suspended to allow the removal of the freezing tubes. On February 14th, 1888, about two months after congelation had been stopped, quicksand burst into the shaft through one of the holes from which the freezing pipe had been withdrawn. Work was at once resumed on the lining, and the remaining tubes were left in place.

Shaft at Chapin Mine.—This shaft is at Iron Mountain, Michigan. The contract for the work was let in 1887, and the shaft was completed in the early part of 1889.

The Chapin Mining Company found it advisable to locate a shaft near the center of a small valley crossing its property. Previous attempts to sink by ordinary means had failed, and a contract was made with the Poetsch-Sooysmith Freezing Company, to sink by the Poetsch system.

The rock and ore beds approached the surface on both sides of the valley, but dipped down under the center, and, at the point where the shaft was located, they were covered by about 95 ft. of loose material. The overlying formation was sand interspersed with layers of gravel and boulders. Many of the boulders were of large size and were packed so closely that they had the appearance of having been laid by a mason. The sand, which was fine, had a little clay mixed with it, and was very unstable. The miners said: "It will run where water will." The level of the ground-water, originally, had been close to the surface in the center of the valley, but pumping at other points had lowered it to a depth of about 10 ft. at the shaft.

Careful experiments showed that the fine sand contained about 17% of water; the proportion in the other materials was not nearly as great. The relations of sand and boulders are shown in Fig. 29. The shaft, originally, was intended to be circular, but was changed to a rectangular form, 15½ x 16½ ft.

The freezing tubes were placed on the periphery of a circle, 29 ft.

PROGRESS DIAGRAM OF SHAFT SUNK AT CHAPIN MINE, IRON MOUNTAIN, MICHIGAN, BY THE FREEZING PROCESS.

From *School of Mines Quarterly*, Vol. XI.

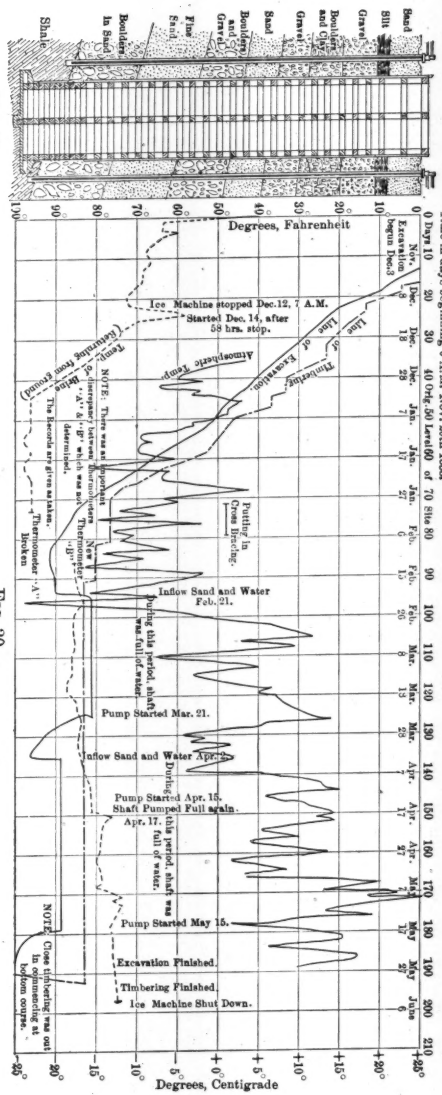


FIG. 29.

FIG. 30.

in diameter. Mr. Moran and *Engineering News* state that twenty-six pipes were used, while Mr. Abbott gives their number as twenty-seven. The bore-holes were cased with 10-in. pipes, and great difficulty was found in keeping them plumb, particularly in passing through the boulders. As soon as the casings had penetrated the ledge, 8-in. freezing pipes were lowered into place and the casings withdrawn. The freezing tubes had flush joints, both inside and outside, were $\frac{3}{8}$ in. thick, and varied in length from 87 to 97 ft. The joints were tested under heavy pressure as the pipe was lowered into position. The inner tubes were $1\frac{1}{2}$ ins. in diameter. The fittings at the top of the freezing tubes are shown clearly in Fig. 1.

A Linde machine, having a refrigerating capacity of 50 tons daily, was used for congelation.

The motive power was an Allis-Corliss engine using compressed air at 60 lbs. pressure and developing 55 H.-P. at sixty revolutions per minute. The freezing medium was a 25% solution of the ordinary chloride of calcium of commerce. About 200 cu. ft. of brine were used, and made a complete circuit every 33 minutes. The temperature of the brine on returning from the tubes is shown graphically in Fig. 30. This diagram also shows the temperature of the atmosphere, which had a very considerable effect on the condensing water used for liquefying the ammonia. The temperature of the brine, as it entered the tubes, was about 1° Cent. lower than on returning to the cooling tank. A test pipe midway between two freezing tubes was frozen on the sixth day after the refrigerating machine was started, and a second pipe, 5 ft. 3 ins. inside the circle of congelation tubes, was frozen on the twenty-first day. Tests of the frozen sand gave an average tensile strength of 431 lbs., and a corresponding compressive strength of 575 lbs., per square inch.

Excavation was started fifteen days after the refrigerating plant was put in operation. On the twenty-third day after freezing began the machine had to be stopped, as the supply of compressed air failed. Congelation was suspended for 60 hours; consequently, excavation and timbering were stopped until the twenty-eighth day. From this time until the ninety-third day, excavation was carried on without any interruption, except for placing timber. For the last thirty days, water had been coming up through the uncongealed core. The water remained clear until the ninety-third day, when it increased in vol-

ume, and sand began to come with it. As the volume of both sand and water increased rapidly, the shaft was at once filled with water, in order to equalize the pressure and thus prevent further flow. An additional freezing pipe was put down, and the leak was closed. On the one hundred and thirty-fifth day, when the bottom of the pit was 7 or 8 ft. in the rock ledge, water and sand again burst into the shaft along the upper surface of the ledge. The water, apparently, finding its way through fissures in the unfrozen rock ledge, had eventually warmed up the rock enough to thaw the joint between the ledge and the sand. As soon as this joint was broken, the water pressure was sufficient to force a current through and disintegrate rapidly the lower portion of the ice wall. It was necessary to flood the shaft again, but, before doing so, a coil of pipes was suspended against the wall near the leak.

This coil was connected with the ice machine, and served effectually to shut off the leak. Excavation was carried on with picks and chisels, and perhaps by the use of explosives. There is a conflict of statements on the latter point. Mr. Thomas states that heavy charges of dynamite were required, while *Engineering News*, in its description, says black powder was used for a time, but was discarded finally, for fear of its effect on the walls.

No temporary lining was used, but the placing of the permanent wooden lining followed the excavation closely. Heavy cross-timbers were set across the top of the shaft, and a rectangular crib was suspended by means of eye-bolts. Other cribs were placed as the work progressed, each suspended from the one above. The cribs were sheathed with planks placed as closely together as possible. The joints between the planks were water-proofed with strips of tar-paper dipped in hot pitch and held in place by battens. Wedges were driven between the lagging and the ice wall to hold the lining in true position, and the space outside the latter was filled as completely as practicable. The joint between the lining and the ledge was made by fitting the timbers to the rock surface carefully and then grouting around them with cement mortar. The lower 15 ft. of lining was close timbered. The construction of the lining is shown clearly in Figs. 31 to 34, inclusive. The progress of both lining and excavation is illustrated graphically in Fig. 30.

After the completion of the work, in order to test the thickness of the ice wall, a pit was sunk in the outer edge of the wall for a

depth of 20 ft., and a steel rod was driven down considerably further. The wall had a batter of about 1 in 15. The diameter of the ice wall at the bottom of the pit was 54 ft. Water did not find its way through the ice wall until fifty days after shutting down the refrigerating plant.

The alignment of the lining was not disturbed by the melting of the ice wall.

Jessenitz Mines.—These mines are near Lübtheen, in Mecklenburg-Schwerin.

Work, by the congelation process, was commenced in 1887, and abandoned about 1891. The shaft was not completed until 1902. When the work was started, it was decided to sink through the first 262.5 ft. by congelation; below this depth, it was thought the measures were so compact that ordinary means would suffice. The plans were carried out successfully to a depth of 410.1 ft., when water-bearing measures were encountered again, and, as a bore-hole revealed the fact that they extended to a depth of 590.6 ft., the freezing process was resorted to again.

Mr. M. F. Schmidt states that the material penetrated in the upper portion of the shaft was as follows:

Yellow sand	39.0 ft.	Blue clay	1.6 ft.
Gray sand	21 7 "	Gypsum and sand	7.4 "
Gravel	5.7 "	Fissured gypsum	11.5 "
Rough gray sand	32.0 "	Gypsum	34.5 "
Coarse gravel	4.1 "	Alternations of gypsum, clay and limestone	53.2 "
Clay and gravel	6.1 "	Total	227.1 ft.
Fallen-in rocks	4.6 "		
Limestone	5.7 "		

Chief Engineer Riemer gives the following list of the strata penetrated by the various processes:

Sand	62.3 ft.	Gypsum, hard	243.0 ft.
Mixed gravel, part fine and part coarse	45.7 "	Cavernous gypsum, with clay	12.0 "
Gypsum, with interbeddings of sand	33 0 "	Sandstone, fissured	24.0 "
Gypsum, both hard and soft	82.0 "	Gypsum, with anhydrite	383.0 "
		Total	885.0 ft.
		Rock salt.	

PLANS OF LINING. CHAPIN MINE.

From *School of Mines Quarterly*, Vol. XI.

FIG. 31.

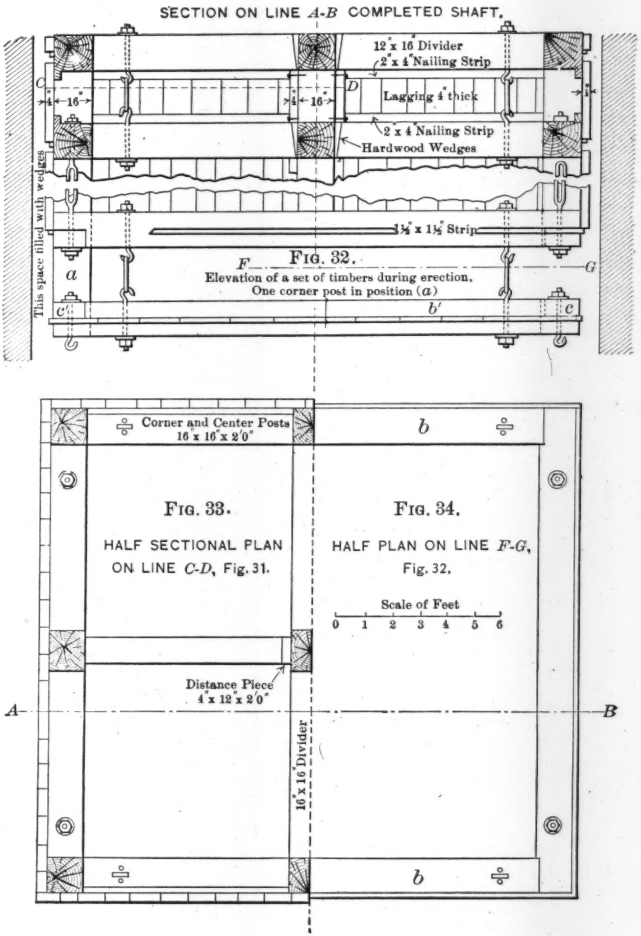


FIG. 32.
Elevation of a set of timbers during erection. One corner post in position (*a*)

FIG. 33.
HALF SECTIONAL PLAN ON LINE *C-D*, Fig. 31.

FIG. 34.
HALF PLAN ON LINE *F-G*, Fig. 32.

The ground-water level was 26.2 ft. below the surface.

The shaft was circular, and 16 ft. 4 ins. in diameter. For a depth of 23.0 ft., a well, with a diameter of 30.4 ft., was sunk by ordinary means.

Twenty freezing tubes were put down on the periphery of a circle, 22 ft. 11 ins. in diameter. Sixteen of these holes were more than 230 ft. deep, one was 328.1 ft., and three were 262 ft. deep. Owing to the difficulty of closing the bottom of the casings, a smaller pipe, with its lower end welded and tested to a pressure of ten atmospheres, was placed inside, to serve as the freezing tube. Fig. 21 shows the arrangements for making the borings, as well as the location of the holes.

Congelation was started on July 26th, 1887. After the machine had been working one hundred and eight days, the diameter of the ice cylinder was 29 ft. 6 ins.

Sinking was begun on November 10th, 1887, and completed to solid material on March 14th, 1888.

The shaft was lined with cast-iron tubbing backed with concrete. The lining was completed on July 5th, 1888. Efforts were then made to thaw the ice tower by filling the shaft with water at 60° Cent., and maintaining it at that temperature for two weeks. After emptying the shaft, sinking was continued in the ordinary way to a depth of 410.6 ft., where the water-bearing strata were encountered again. It was decided to sink twelve tubes, on a circle 33 ft. 4 ins. in diameter, to a depth of 623.4 ft. Although eight months had elapsed since the ice machine was stopped, the frozen measures were encountered, and the attempt to sink the tubes in this position was abandoned. Eight pipes were then put down on the circumference of a circle inside the shaft.

It was proposed to sink first a small shaft of the same diameter as the circle on which the pipes were placed, and then enlarge to the required diameter by excavating outside the tubes. Before sinking the small shaft, the ice wall was formed, apparently without difficulty, but, after the pipes were exposed, they were not able to maintain the necessary thickness of the ice. Mr. Schmidt states: "The pipes once exposed did not possess the same radiating refrigerating power, and the loss of cold inside the shaft was enormous."

On January 13th, 1890, at a depth of 492.1 ft., the ice wall gave way and the sand rose 16.4 ft. in the shaft. Freezing was continued for thirteen months without result. The attempt to sink by means of congelation was abandoned finally and the Kind-Chandron method adopted.

The failure of the freezing process during the latter part of the work is attributable partially to pumping, which started an underground current containing potash salts. These salts lowered the congelation point of the material to be frozen.

Georgenberg Mine.—This mine is near Tarnowitz, in Upper Silesia, Germany.

The sinking was done during 1890 and the early part of 1891.

The object of the shaft was to obtain access to a bed of iron ore. Previous to adopting the freezing process, an attempt was made to sink by pumping, at a point 328 ft. from the site finally selected, but had to be abandoned. The size of the shaft was 9 ft. 10 ins. x 14 ft. 8 ins.

The material to be penetrated consisted of about 78 ft. of tertiary strata, described as quicksand, and containing a great deal of water. Underlying the tertiary strata was a bed of limestone.

The boring, freezing and lining required nine months.

No. 10 Shaft, Lens Colliery.—This colliery is at Vendin-le-Viel, Department of Pas-de-Calais, France.

Most of the work was done during 1891. The first attempt to sink was made with a masonry caisson. A masonry tower, 53 ft. high, was constructed on a cast-iron shoe or cutting edge. Excavation was carried on inside the tower, and the work was kept dry by pumping. The tower stuck fast, finally, at a depth of 41.0 ft. Sinking was then continued with sheet-piles to a depth of about 80 ft. At this point the temporary lining gave way, partially; the foundations of the permanent plant, which had been built around the shaft, began to settle, and a 1 000-H.-P. pumping plant was unable to control the water. Under these conditions, preparations were begun to continue the work by means of congelation. The shaft was circular, with a clear diameter of 15 ft. 8 ins.

The nature of the material, from the surface down, is shown by the following:

Vegetable earth	1.6 ft.	Yellowish chalk	3.9 ft.
Clay, with shifting marl	2.6 "	Fissured chalk, with flints	49.8 "
Sandy clay	3.6 "	Clayey chalk	1.3 "
Clay, with reddish earth	3.0 "	Soft bed	0.7 "
Plastic clay	1.3 "	Very hard bed	4.6 "
Coarse-grained sand	5.1 "	Hard bed	3.3 "
Shifting, clayey sand	4.8 "	Hard white chalk	9.0 "
Shifting, green marl sand	6.1 "	Millstone grit	2.1 "
Marly sand, with broken chalk	3.0 "	Bleus (a blue impure limestone)	2.8 "
Green, marly sand	1.6 "	Non-aquiferous clayey marls	5.7 "
Shifting, clayey chalk	3.3 "	Gray and blue chalk	124.6 "
Rich chalk	6.9 "	Marl (dieve)	1.6 "
Soft chalk	6.1 "	Total	265.1 ft.
Clayey chalk	4.1 "		
White chalk	2.6 "		

The natural water level was at a depth of 15 ft., and the last three strata in the list may be said to be impermeable.

Owing to the foundations around the top of the shaft, it was necessary to space the freezing tubes quite irregularly. Eight tubes were put down inside the shaft, on the circumference of a circle and as near the lining of the shaft as practicable. Outside the shaft, twenty tubes were spaced, as regularly as conditions would permit, over a square of about 25 ft. on a side. The lining of the bore-holes was 14 ins. in diameter at the top, 10¼ ins. near the middle, and 8 ins. at the bottom. In the bore-holes, 6-in. freezing tubes, 137.8 ft. long, were placed, and the linings withdrawn. The circulating tubes were 2 ins. in diameter.

Congelation was begun on April 9th, 1891, with an Osnabrück machine driven by a 40-H.-P. engine, and required two hundred and three days. Water stood to a depth of 50 ft. in the shaft during freezing. Mr. Remaux, Engineer of the work, calculated that the congealed beds contained about 40% of water. The freezing solution was a 20% solution of calcium chloride. The temperature of the brine as it left the refrigerating machine was —7° Cent. at the beginning and —16° Cent. at the end of the congelation period, and was 4° warmer on its return. Just previous to starting excavation, the eight

pipes were disconnected from the freezing machine, and they were removed as the work progressed.

Freezing, however, was kept up in the outer tubes until near the completion of the work. Sinking was effected without temporary lining. Compressed powder was used for loosening the frozen material. The temperature in the shaft was about 2° Cent. when the men were working.

The lining consisted of iron rings, 4 ft. 10 ins. high, each composed of ten segments. The web was $1\frac{3}{8}$ ins. thick at the top, $1\frac{19}{32}$ ins. thick at the bottom, and was reinforced by internal ribs. The flanges were $3\frac{11}{32}$ ins. wide. The iron lining was backed by 18 ins. of concrete. To the mixing water was added 1% of caustic soda. The concrete, although frozen, was found to have set very satisfactorily. A few rings of cast-iron lining were placed below the masonry by the original method, before the work was suspended. On resuming work, the three lower rings were found to have been bent into an oval form, and they had to be removed.

No. 10, Bis Shaft, Lens Colliery.—No. 10, bis shaft, is 100 ft. from the No. 10 shaft, and was sunk during 1892.

The work followed that on the No. 10 shaft, and, from the first, congelation was decided on. A pit, 20.3 ft. in diameter, was sunk to ground-water level, a depth of 11.5 ft., by ordinary means, and lined with brick masonry. From the bottom of the pit the bore-holes were started. The finished diameter of the shaft was 12 ft. The material penetrated was similar to that of Shaft No. 10. Twenty holes were put down on the circumference of a circle, 16.9 ft. in diameter, and four holes on the circumference of a circle, 7.1 ft. in diameter, to a depth of 137.8 ft. The sinking of the pit at the top occupied twenty-five days; the borings and the placing of the refrigerating tubes occupied fifty-six days. After placing the freezing tubes, the linings of the bore-holes were withdrawn. The same machine was used for congealing as at No. 10 shaft. Freezing was begun on June 8th, and was continued for seventy-five days. Near the end of the work, the refrigerating liquid had a temperature of —20° Cent. on entering the freezing pipes, and —17° Cent. on leaving them.

Excavation was begun on August 16th, and was completed on October 1st. Near the top, the excavation was made only a little larger than the finished diameter, but its size was increased toward

the bottom. Some powder was used, and the shaft was lined temporarily. The alternating bands of sand and clay proved to be somewhat unfavorable, as the clay froze much more slowly and was less hard than the other strata. Where the clay was plastic it assumed a shaly structure, and its difference in behavior was likely to break the freezing pipes and produce cracks in the ice wall. The cracks, in this case, however, did not result in leaks, as any water which found its way into them was congealed before it reached the face. As soon as the excavation reached the required depth, the permanent lining was placed, working from the bottom up. The lining consisted of cast-iron rings, $1\frac{25}{32}$ ins. thick, of six segments each.

No. 3, Bis Shaft, Dourges.—This shaft is located in the Department of Pas-de-Calais, France, and was sunk during 1892 and 1893.

An attempt was made to sink by pumping out, but, at a depth of 62.2 ft., the three pumps, working at their full capacity, 80 cu. ft. per second, were still unable to control the flow. At this stage it was decided to adopt the Poetsch system, and the excavation already made was filled with sand.

The kinds of material to be penetrated were as follows:

Vegetable earth	1.3 ft.	Broken chalk	17.7 ft.
Clay	3.3 "	Harder chalk	8.5 "
Sandy clay	1.0 "	Chalk	12.8 "
Fine-grained limestone	1.1 "	Chalk, in thicker layers	10.2 "
Yellow sand	0.9 "	Chalk, with flints	3.5 "
Fine-grained limestone, with gravel	2.8 "	White chalk	16.4 "
		Chalk, in thick layers	9.0 "
Sand	1.5 "	Gray chalk	14.9 "
Sand and gravel	3.1 "	Chalk, with flints	7.6 "
Broken chalk	8.2 "	Sandstone, with flints	16.6 "
Sandy clay, with gravel	9.5 "	Sandstone, with much chalk	17.4 "
Sandy earth, with much gravel	9.8 "	Millstone grit	11.1 "
Sandy earth, slightly more compact	9.8 "	Total	198.0 ft.

The natural water level was at a depth of 23.3 ft. The shaft has a clear diameter of 15 ft.

The freezing fluid was circulated through twenty-four freezing tubes on the circumference of a circle 21 ft. in diameter. There was

also a tube in the center of the shaft. The freezing tubes were sunk through the millstone grit and 8 ft. into the limestone below.

Work on the borings was started on October 5th, and was completed on December 21st. After placing the freezing tubes, the lining of the bore-holes was withdrawn.

Congelation was started on January 28th, 1893, with a Fixary machine capable of producing 1 ton of ice per hour.

Sinking was begun on March 27th. The sand was found to be damp, down to a depth of 36.9 ft. Below that depth the measures were entirely frozen. Below the bottom of the previous excavation compressed powder was used. The work was carried on in four 6-hour shifts, there being six drillers and three shovelers in each shift. The rate of progress was about 5.3 ft. per day. At a depth of 105.2 ft., which was reached in May, the material was found to contain very little moisture and to be in an unfrozen condition. At the bottom of the freezing tubes the ground at the center of the shaft was found to be frozen 1.3 ft. deeper than at the sides. During the progress of the excavation the shaft was lined temporarily with oak cribbing. At a depth of 42.7 ft. excavation was suspended and a cast-iron lining placed, extending above the water level. Cement, mixed with water containing carbonate of soda, was used to back the lining.

The time required for the various operations was as follows:

Boring	76 days.
Erecting cold-producing plant	35 "
Congelation	60 "
Sinking	73 "
Placing tubing	36 "
Total	280 days.

Dourges No. 7, Shaft.—This shaft was sunk during 1894 and 1895. As this work was started after the completion of the No. 3, bis shaft, it was decided to use the freezing process from the first. The first operation was to sink, presumably to water level, a circular shaft, 27 ft. in diameter and lined with masonry.

The shaft was 15½ ft. in diameter. The materials above the millstone grit are 190.2 ft. thick. The water level was at a depth of 26.2 ft.

Twenty freezing tubes were put down on the circumference of a circle 22 ft. in diameter. A single hole was also put down in the center and insulated so as to concentrate the freezing effect at the bottom of the shaft. As at No. 3, bis shaft, the lining tubes of the bore-holes were withdrawn. The tops of the freezing tubes were at a depth of 13.1 ft.

Two Fixary machines, each capable of producing 1 000 lbs. of ice per hour, were used for congelation.

Flines-les-Raches.—This shaft is located in the Department of Le Nord, France, and was sunk mainly in 1894.

The nature of the material penetrated was as follows:

Vegetable earth	1.6 ft.	Argillaceous sand	1.6 ft.
Peat	2.0 "	Fissured chalk	11.5 "
Bluish clay	3.0 "	Compact chalk	6.6 "
Very fine-grained sand	26.0 "	Fissured white chalk	6.6 "
Gray argillaceous sand, with shells	12.5 "	Compact chalk	14.8 "
Very hard gray sand	6.2 "	White chalk	16.4 "
Plastic clay	13.1 "	Grayish chalk	6.6 "
Sandy clay	3.3 "	Chalk	62.3 "
Very dry sandy clay	10.2 "	Chalk, with flints	23.6 "
Fine-grained sandstone	2.3 "	Chalk and sand	7.9 "
		Total	238.1 ft.

The shaft is circular, and is 13 ft. 8 ins. in diameter.

Twenty-one freezing tubes were put down, to a depth of 246.1 ft., on the circumference of a circle, and a twenty-second hole was located in the center of the shaft. At a depth of 85.3 ft., a spring was encountered which gave rise to a strong flow in the bore-holes, rising 6.6 ft. above the surface of the ground. This flow was increased by another stream at a depth of 219.8 ft.

This current of water ascending along the freezing tubes would have made freezing very difficult. In order to stop it, two masonry walls were built around the top of the shaft. These walls were built to such a height as to be above the piezometric level of the water. The stream was also tapped by a bore-hole 82 ft. distant from the shaft. By these means, the level of the water was maintained at a height of 4.9 ft. above the ground. The freezing tubes were prolonged to a height of 8.5 ft. above the surface of the ground, in order to be well above the water level.

The machine used for congelation was of the Fixary type, and had a capacity of 2 200 lbs. per hour. Freezing was begun on September 1st, 1894, and in thirty-eight days the ice tower was united. The freezing medium was a solution of calcium chloride.

Excavation was begun on October 25th, and carried on mainly by picks and wedge needles. Near the bottom, however, compressed powder was used. The central tube was dismantled as the work of sinking progressed.

The lining consisted of oak cribs, about 7 ins. thick, and was placed in sections of 45.9 ft., working from the bottom of each section upward. The timber was backed by 8 ins. of concrete. The lining was completed on May 1st, 1895.

The cost per foot was as follows:

Congelation	$91.17
Sinking	8.82
Tubbing	38.23
Concrete	2.94
Total	$141.16

Vicq Pits, Anzin Colliery.—This colliery is in the Department of Pas-de-Calais, France. Most of the work was done during 1894.

The Anzin Company desired to sink two shafts through what was known to be very treacherous material, and, after considering carefully the various methods, decided to adopt the Poetsch process. Work on the two shafts was carried on simultaneously. The two shafts are circular in plan, and 121 ft. apart, from center to center. The clear diameter of the larger, used for winding, is 16 ft. 4 ins., and the diameter of the smaller, used for pumping, is 12 ft.

A bore-hole indicated that the strata to be passed through were as follows:

Vegetable earth	3.3 ft.	White marl	73.2 ft.
Greenish sand	5.6 "	Rich marl	42.9 "
Blackish sand	1.3 "	Marl and pyrites	4.6 "
Gravel	11.9 "	Sandy marl	8.5 "
Sand	11.5 "	Compact limestone	4.9 "
Sandy clay	1.6 "	Flints	41.0 "
White chalk	30.3 "	Impure blue limestone	36.4 "
Marl	31.5 "	Total	334.7 ft.
Gray marl	26.2 "		

The strata were water-bearing, down to a depth of 298.6 ft., but there existed two distinct levels: one at the surface varying with the rainfall, and capable of supplying about 6 or 7 cu. ft. per minute; the other, in the upper and badly fissured chalk layers, was under such pressure that the water rose about 2½ ft. above the surface as soon as an opening was made.

As a preliminary to starting work, a well, about 800 ft. distant from the site of the pits, was sunk to the chalk, for supplying feed and condensing water. About 3 500 galls. per hour were needed. The well was 6½ ft. in diameter, and 49 ft. deep, and cost about $2 000. The freezing tubes were put down on the circumferences of circles the centers of which were at the centers of the respective shafts. Twenty tubes were used for the larger shaft and sixteen for the smaller one. The diameters of the respective circles were 21 ft. 3 ins. and 16 ft. 7 ins. The freezing tubes were $4\frac{1}{2}\frac{9}{2}$ ins., inside diameter, $\frac{9}{32}$ in. thick, and 298.6 ft. long. The circulating tubes were $1\frac{3}{16}$ ins., inside diameter, and $\frac{5}{32}$ in. thick.

As mentioned previously, water flowed from the bore-holes (an 8-in. hole yielding 20 cu. ft. per minute), and in order to prevent loss of heat, it was necessary to cut off this flow. Small wells were sunk around each hole, and wrought-iron pipes, 95 ft. long, and 10 ins. in diameter, were set around each freezing tube. The tops of these pipes were above the piezometric level of the water, and the wells around the pipes were then filled with cement. The joints of the refrigerating pipes were tested to a pressure of 284 lbs. per square inch. Apparently, more than usual care was taken in making all the connections, not only to secure tightness, but to reduce friction as well, and to that end the collecting rings were made 8 ins. in diameter. The pits were covered with sheds with quite complete arrangements for handling the pipe and making the borings. The sheds, afterward, were modified for use in sinking.

The problem of congelation was very carefully studied, and the calculations, in considerable detail, are given in a paper by Saclier and Waymel.* It is estimated that the measures for the first 92 ft. contained 50% of water, and that the remaining 207 ft. to be congealed contained 25 per cent. The required thickness of the ice cylinder was found to be about 1 ft. 4 ins. To produce the desired

*Bulletin de la Société de l'Industrie Minérale, 1895, p. 27.

result, it was calculated that 243 106 041 pound-calories must be removed from the ground around the larger shaft, and 191 816 799 pound-calories from around the smaller one. Allowing 25% for loss, the total to be removed was 543 653 550 pound-calories. The time allowed for congelation was one thousand hours. Accordingly, a machine capable of removing 543 700 pound-calories per hour was required. The plant installed was driven by a 200-H.-P. engine, and consisted of four Linde machines, coupled two together, and with a combined capacity of 4 tons of ice per hour. The freezing solution consisted of 30 tons of calcium chloride dissolved in 2 472 cu. ft. of water. The rate of circulation was 70 6 cu. ft. per minute.

Careful preparations were made to record the performance of the machines and to observe the progress of the freezing. To accomplish the latter, on the circumferences of circles, 27 ft. 9 ins. in diameter for the larger, and 23 ft. 3 ins. in diameter for the smaller pit, and radially opposite the freezing tubes, 2-in. pipes, 6 ft. 6 ins. long, were driven. These pipes were filled with a solution of calcium chloride, and their tops were protected from the effects of the atmosphere. Thermometers, in cases arranged so that they could be read without removing them from the solution, were lowered to the bottom.

On a line joining the centers of the two shafts, similar tubes were placed every 13 ins., thus forming a scale for noting the advance of congelation. The freezing apparatus was started on May 28th, 1894, but the four machines were not in full operation until June 12th. With all four compressors running, the temperature of the liquid on leaving the compressor was —15° Centigrade. It was from 2° to 3° higher on returning to the machine. On July 1st, the thermometers showed that the thickness of the ice on the outside of the freezing tubes was 1 ft. 5 ins., and on the inside 2 ft. 5 ins. Only two or three of the freezing machines were run after that date. On July 16th, the ice was 1 ft. 9 ins. thick on the outside and 3 ft. 3 ins. thick on the inside of the ring of pipes. On July 15th, the initial freezing operations were considered to have ended. The thermal equivalents of the work done during this period are given in Table No. 2.

It is stated that there was 20% loss in the cooling effect at the surface of the ground, owing to the distance of the machines from the shafts.

Excavation in the smaller pit was started on July 2d, and in the larger pit on July 16th. The larger shaft was excavated to a diameter

of 18 ft. 5 ins. It was lined temporarily to a depth of 100.7 ft. At that depth excavation was suspended, and the permanent lining was placed, from the bottom upward.

TABLE No. 2.

	Heat Absorbed, in Pound-Calories.	
	Smaller pit.	Larger pit.
Formation of ice	94 885 984	154 487 786
Cooling ground, outside circuits	37 093 751	62 358 974
" " inside "	31 909 341	50 524 714
Total utilized	163 889 076	267 371 474
Loss	56 381 851	72 297 511
Totals	220 270 927	339 668 985
Work of engines	221 297 073	355 723 015

Excavation at the bottom of the freezing tubes was completed on October 16th. No temporary lining was required for the second portion of the work. After lining the second section permanently, excavation, through uncongealed material, to the coal measures, was completed without incident. Below the depth of 100 ft., artificial ventilation was necessary. The temperature of the shaft lowered gradually during excavation; at first it was +2° to +3° Cent., at the completion of the first section 0°, and near the bottom —12° Centigrade. The workmen, however, who were employed in 8-hour shifts, suffered no inconvenience from the low temperature. During the first period the area to be excavated was only frozen slightly, and very rapid progress (6 or 7 ft. per day) was made.

The ice wall increased in thickness gradually, but a fair rate of progress was maintained to a depth of 257.5 ft., where the flints were encountered on September 15th. In this material, the thickness of the ice wall was much greater, although there was still an unfrozen core. The thickness of the ice wall varied with the material, and was much less in the fine sand than in the coarse gravel, notwithstanding the larger proportion of water in the latter. In the fissured chalk, blocks of ice, containing as much as 1 cu. yd., were encountered. The frozen conglomerate of flint was very difficult to break up. The

tools were broken and the points dulled very rapidly. As many as three thousand picks and wedges had to be sharpened in a single day. In clay, sinking proceeded rapidly, and the frozen mass had the consistency of puddle cemented with ice. At the bottom of the freezing tubes, 298.6 ft. below the surface, the uncongealed core still had a diameter of 3.3 ft. The ice was found to extend 2.8 ft. below the freezing tubes. The history of the excavation in the smaller shaft is similar to that of the larger, except that, owing to its smaller diameter, the entire mass became frozen through the flints.

At the time of the completion of the excavation to the bottom of the freezing tubes, observation showed that the thickness of the frozen mass outside the tubes was about the same at the top and bottom, and the engineers in charge state that the frozen portion was roughly a cylinder. Both shafts, for a depth of 386 ft., were tubbed with cast-iron rings, 1 in. thick at the top and $1\frac{25}{32}$ ins. thick at the bottom of the shaft. Concrete, 8 ins. thick, was placed behind the tubbing. In the main, this was composed of two-fifths very hydraulic lime and three-fifths calcined coal measure shale. To the water, 10% of calcium chloride was added.

The cost is summarized as follows:

Patent royalty, for two shafts, each 298.6 ft. deep, $10.59 per foot	$6 324.35
Temporary installation	3 779.33
Boring	14 218.89
Refrigerating machine, including freezing pipes, collector-rings, etc	48 009.91
Experimental apparatus	366.70
Congelation, May 28th to December 28th	6 374.98
Sinking and tubbing	55 478.82
Miscellaneous	2 448.01
Total	$137 000.99

The cost is given by Messrs. Saclier and Waymel as $177.46 per foot of shaft. This, evidently, is for the 386 ft. of tubbed shaft, of which a depth of 298.6 ft. was in congealed material.

In the paper by Messrs. Saclier and Waymel, the engineers, several interesting conclusions are given, and these may be summarized as follows:

The bore-holes must be absolutely plumb. For depths not exceeding 328 ft., they may be spaced 4 ft. apart. For depths of 656 ft., they should not be more than 3.3 ft. apart. The radial distance of the tubes from the center of the shaft should not exceed the radius of the shaft by more than 2 ft. Freezing tubes should not be placed inside the shaft, for two reasons:

1.—Excavation is much more difficult when the entire mass is frozen.

2.—Bodies of water are likely to be imprisoned between the inner mass and the annular ice wall. The advance of congelation will then produce pressures which may rupture the ice walls. The refrigerating pipes should be tested carefully, and should be of a metal as soft as practicable. The collecting rings should be of large cross-section, and all connections with circulating pipes should be made in uniform manner.

No. 9, Shaft, Courrieres Colliery.—This colliery is in the Department of Pas-de-Calais, France.

The sinking was practically accomplished in 1894. An attempt was made to sink this shaft by pumping. At a depth of 102.5 ft., three pumps, lifting 8.5 cu. ft. per second, could not handle the water. As there was no room for additional pumps, this method was abandoned, and the Poetsch system adopted.

The material penetrated was as follows:

Vegetable earth	0.7 ft.	Gray marl	23.3 ft.
Yellowish clay, mixed with chalk	9.2 "	Gray marl, with flints	6.9 "
Blue potter's clay	1.6 "	Gray-yellowish marl	4.9 "
Sand, mixed with potter's clay and marl	11.5 "	White marl	9.8 "
Fine marl	3.3 "	Gray and stronger marl	2.1 "
Fine marl, with flints	78.6 "	Millstone grit	12.6 "
		Impure blue limestone	1.0 "
		Total	165.5 ft.

The last two strata were impermeable.

The shaft was circular in plan and 15 ft. in diameter. Twenty-four pipes were put down on the circumference of a circle 22 ft. in diameter. A twenty-fifth pipe was sunk in the center and insulated so as to freeze only the material near the bottom. The refrigerating pipes were 171.5 ft. long, and their internal diameter was $4\frac{17}{32}$ ins. The diameter of the circulating pipes was $1\frac{5}{16}$ ins.

The freezing plant consisted of two Fixary machines, of 1 100 lbs. capacity per hour. The freezing solution was calcium chloride.

Congelation was started on April 7th. Excavation was commenced on June 25th, and was completed on July 17th.

The lining was of timber backed by concrete. The water used for mixing the concrete contained a solution of carbonate of soda to prevent freezing.

To complete the work, two hundred and twelve days were required, distributed as follows:

Boring wells and lowering brine pipes	102 days.
Erecting refrigerating plant	10 "
Freezing	78 "
Sinking, excavation	22 "

Venus Tiefbau Colliery (Winding Shaft).—This shaft is at Brux, Bohemia. The construction required from January, 1895, to June 1st, 1896.

The coal beds of this part of Bohemia are covered by a thick bed of quicksand, and several attempts to penetrate it at different points have met with failure. One instance is cited where the quicksand broke into and nearly filled a mine. The owners of the Venus Tiefbau Colliery first attempted to sink by ordinary means, and to line the shaft with masonry. As the quicksand was approached, it burst into the shaft and filled it to a height of 72.2 ft. A depth of 19.7 ft. was gained, after pumping out, but a second rush stopped work by this method. Next, an attempt was made to sink a masonry lining built on piles by excavating from the bottom. After advancing about 26.2 ft., there occurred a third break of such violence as to throw the masonry out of plumb, and put a stop to work by this plan. At that stage the Poetsch process was adopted.

The nature of the material encountered was as follows:

Clay	105.0 ft.	Quicksand	23.0 ft.
Quicksand	72.2 "	Clay	196.8 "
Clay	59.1 "	Total	456.1 ft.

The pressure in the upper layer of quicksand was two atmospheres, and in the lower layer, from seven to eight atmospheres.

The clear diameter of the shaft was 13 ft. 5 ins. Twenty-four holes were put down on the circumference of a circle having a diameter

of 26 ft. 2 ins. The freezing tubes were placed inside the lining of the bore-holes. Instead of having their lower ends welded, the freezing tubes were closed at the bottom with a screw-plug which could be removed from the top. This provision was made in order to allow the escape of the freezing liquid upon the completion of the work. It was thought that the brine would help to thaw the pipes free and aid in their withdrawal.

The borings were started in the middle of January and completed about April 15th.

The refrigerating apparatus was of the Carré-Kropff type, and consisted of three machines. One had a capacity of 1 ton of ice per hour, and each of the others a capacity of half a ton. Seldom more than two of the machines were used at a time. Congelation required from June 1st to December 2d, 1895.

Sinking was commenced on December 2d, and progressed at the rate of about 3.3 ft. per day. At a depth of 108.3 ft., some water entered the shaft, and sinking was interrupted until the openings were completely frozen. Below a depth of 173.9 ft., the entire central mass was congealed. The frozen quicksand had the compactness of soft sandstone, and was excavated with picks and wedges. Only a light temporary lining was required.

After passing the lower quicksand bed, on February 10th, 1896, the permanent lining was begun. Except through the quicksand, the lining was of masonry. In the lower strata, iron tubbing was used, and in the upper strata dove-tailed concrete blocks. On May 15th, 1896, the lining was completed and the freezing machine stopped. The shaft was filled with water at a temperature of 50° to 60° Cent. to assist the setting of the mortar and to thaw the ice wall. On pumping out the shaft, it was found to be practically tight.

Prof. Petrilk gives the total cost of lining and sinking at about $140 per foot.

It is stated that the air-shaft, also, was sunk by congelation. This shaft was about 130 ft. from the winding shaft.

Ligney-Les-Aire Shaft.—This shaft is in the Department of Pas-de-Calais, France. The work was done during 1895 and 1896.

The thickness of the permeable covering to be penetrated was about 270 ft. The borings and the installation of the freezing machinery occupied the time from the end of April to October 12th, 1895. Congelation was started on December 19th, producing a temperature

of —5° Centigrade. The temperature of the ground at this time was +12°.

Sinking was begun on February 19th, 1896, and on May 6th the end of the frozen ground was reached, at a depth of 300 ft.

The lining was cast-iron tubbing, in rings 5 ft. high, consisting of eight segments. The tubbing was founded on a timber crib at the bottom of the shaft. The lining was completed on July 16th, 1896.

Auboué Iron Mine.—This mine is in the Department of Meurthe-et-Moselle, France. Operations were begun in 1898, and the shaft was completed in 1900.

The construction of other shafts in the vicinity, by different methods involving pumping, proved to be very slow and costly. Accordingly, it was decided to adopt the Poetsch process.

The finished diameter of the shaft is 16.4 ft., but, to allow for placing the lining, it was excavated to a diameter of 18.5 ft.

The material penetrated was as follows:

Soft, shelly yellow limestone	31.2 ft.	Crystalline limestone	9.8 ft.
Alternating soft yellow limestone and sandy marl	4.9 "	Crystalline limestone	16 4 "
Compact limestone, alternating with chalk	29.5 "	Soft yellow limestone	14.8 "
Very hard gray limestone,	44.3 "	Gray limestone	6 6 "
Very hard white granular limestone	14.7 "	Ferruginous limestone	6.6 "
Broken crystalline limestone, intercollated in rare places with chalk	32.9 "	Hard limestone and marl.	11.5 "
Soft yellow limestone, alternating with crystalline limestone and hard sand	42.7 "	Ferruginous limestone	9 8 "
		Gray marl	9.8 "
		Ferruginous limestone	3.3 "
		Ferruginous marl	9.8 "
		Pure gray micaceous marl	39.4 "
		Total	338.0 ft.

As a preliminary to making the borings, a well, 24 ft. in diameter, was sunk to ground-water level, a depth of about 20 ft., and lined with masonry. It was first planned to use twenty freezing pipes, located on the circumference of a circle 21.4 ft. in diameter. Many of the

bore-holes, however, deviated from the perpendicular, and it was necessary to reject those which approached the center of the shaft. Those which diverged from the center of the shaft, or from each other, were accepted, provided the deviation was not too great. In all, thirty-one holes were bored, of which twenty-four were used for congelation. The diameter of the first borings was 8 ins., and of the later ones 12 ins. The distance of the latter from the center of the shaft was also increased to 11.5 ft. The 12-in. holes were found to be much more nearly vertical than the 8-in. holes. The freezing and circulating pipes were 4.8 ins. and 1.3 ins. in outside diameter, and 0.2 and 0.117 in. in thickness, respectively. The depth of the holes was 459.3 ft. The tops of the freezing pipes were just above the bottom of the well. One hole was bored in the center to relieve any pressure which might be occasioned by variable rates of freezing in the different strata.

Congelation began on March 29th, 1899, with two Fixary machines, each having a capacity of 2 200 lbs. of ice per hour. Each machine was driven by an 80-H.-P. engine. In general, the temperature of the descending brine was —17° Cent., and of the ascending —12° or —13° Centigrade. Congelation required one hundred days, and, after excavation was started, on July 6th, only one machine was operated. To facilitate excavation, a hoisting house was built over the shaft, and an annular lining was placed in the well to shut off the tops of the freezing pipes from the main portion of the shaft. Black powder was used at first, but, later, dynamite was adopted, with much better results. At the minimum temperature, the pipes contracted 2.4 ins., and those which were gripped so tightly by the surrounding material that they could not move were ruptured at the joints. When a leak occurred opposite the unlined portion of the excavation, the pipe was uncovered by means of a small tunnel, and a water-tight gasket or collar was placed around the break. When the breaks could not be reached in this way, a third pipe, 2.75 ins. in diameter, was placed in the hole, and served as the freezing tube. The rate of progress of the excavation varied from 10 ins. per 24 hours in hard rock to 6½ ft. in micaceous clay.

Three shifts were employed, each consisting of one foreman, seven miners and seven helpers. The average rate of excavation, for actual working time, was 23 ins. in a 24-hour day. Very little temporary lining was required. The shaft below the well was dug in six sections,

From *Le Genie Civil*, 1901-1902
To Accompany Description of Auboué Shaft.

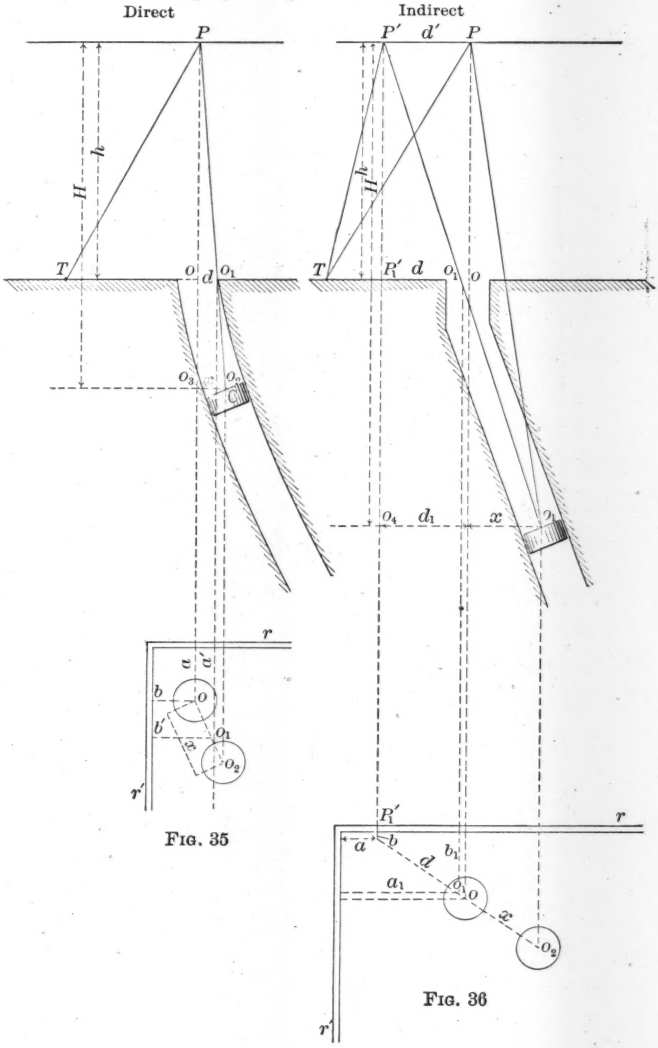


FIG. 35

FIG. 36

of which the upper five, aggregating 368 ft., and passing wholly through water-bearing strata, were lined with cast iron backed by concrete. The sixth section was lined with masonry. In order to shut off percolation, at depths of 338 and 394 ft. below the surface, masonry rings enclosing the freezing tubes were built around the shaft. When the freezing pipes were uncovered to place the lower ring it was found that one of them, 18-*a*, was not coated by frost. This indicated that circulation in that pipe had stopped. At about the same time one of the adjacent tubes was pierced by a drill and had to be disconnected temporarily. Difficulty in locating two of the freezing tubes caused a delay of a week in completing the masonry ring. At the end of that time, water burst into the shaft, near the tube 18-*a*, flowing at the rate of about 6 cu. ft. per minute. For several hours, efforts were made to close the break, but the flow increased gradually and compelled the workmen to abandon the shaft. In order to prevent the wearing away of the ice wall by this flow, water was pumped in from the outside until all flow through the break was stopped. To hasten congelation, the second freezing machine was started, but, a break occurring in one of the pipes, all the brine was lost and congelation had to be suspended. Before the leak could be located, it was necessary to remove all the inner tubes. These were second-hand, and had been corroded badly by long exposure to the brine. Some of them were broken, and much time was lost in recovering the lower portions. The scale or rust from the pipes had fallen to the bottom of the freezing tubes, filling them in some cases to a depth of from 3 to 6 ft. It became necessary to shorten the inner tubes by corresponding amounts before they could be replaced. Altogether, fifty-three days were lost on account of the break in the ice wall.

The sinking and lining were completed on July 5th, 1900.

Harchies Shaft, Blaton Colliery.—This shaft is in Harchies, Belgium. The sinking was accomplished during the years 1899 and 1900.

The shaft is 787 ft. deep, and, on account of the nature of the material encountered, Mr. Saclier, who prepared plans for the work, considered some special precautions necessary. The trial boring, Fig. 14, showed that, between the depths of 76.6 ft. and 168.0 ft. below the surface, there existed a bed of impermeable clay, and, directly beneath this, 575.8 ft. of water-bearing material, mostly greensand. The specific heat of the clay was said to be not more than 0.2, while

that of the greensand, owing to the large quantity of water it carried, was thought to be much higher. Under these conditions, it was probable that the clay within the ring of the freezing tubes would be frozen to the center, while beneath there would exist a cylinder of uncongealed sand and water surrounded by an annular ring or wall of frozen material. The further thickening of the ice wall toward the center would give rise to pressure in the central cylinder, which would no longer be relieved by the escape of the water upward through the clay. It is conceivable that this pressure might become great enough to flatten or crack the freezing tubes, or even rupture the ice wall itself. In order to guard against this contingency, a central tube was put down to the bottom of the greensand to form a means of escape for the water under pressure by rising and flowing off at the surface. To prevent the tube from being closed by freezing through the clay, a steam pipe was provided whereby it could be warmed down to the bottom of the clay. It was also thought that by observing the pressure of the water in the tube a fair idea could be obtained of the progress of congelation. Although the shaft has been completed, information is not at hand to show whether these theoretical conditions existed in actual practice.

The bore-hole previously mentioned penetrated the following formations:

Vegetable earth.....	1.6 ft.	Greensand.....	246.9 ft.
Chalk	26.6 "	Clayey greensand.....	60.7 "
Glauconiferous marl...	4.8 "	Greensand.....	206.0 "
Chalk.....	14.3 "	Clayey greensand.....	33.0 "
Blue impure limestone.	29.2 "	Soft yellowish sand....	23.4 "
Impermeable clay.....	91.4 "	Coal measures.....	328.1 "
Tourtia.....	5.7 "	Total.....	1 071.7 ft.

The strata below the impermeable clay and above the coal measures contained water under sufficient pressure to produce a flowing well with a head of 16.4 ft.

Fig. 14 shows the freezing pipes surrounding the shaft.

The freezing machine was rented from the Anzin Company, and was the same as that used for sinking the Vicq pits.

Congelation was commenced on May 16th, and sinking on August 1st, 1899. Several of the freezing tubes were ruptured, and, instead

of trying to repair them, the inner tube was withdrawn and replaced by two small pipes connected at the bottom. This arrangement worked very satisfactorily.

The excavation was completed on October 24th, 1900. When freezing was discontinued, the inner tubes were withdrawn and the freezing tubes were filled with concrete.

The lining was placed in sections of 164 ft. each. At the bottom of each section was formed a ledge supporting cribs, from which the lining was started. The latter was in rings, 4 ft. 10 ins. high, cast in four segments. While the lining was being placed, excavation was in progress in the next section below.

The Poetsch process was adopted for sinking Shaft No. 2, in the same locality.

Maria Shaft.—This shaft belongs to the Maria Mine, near Aachen, Germany, and was sunk during 1899 and 1900.

It was designed for both hoisting and ventilation, and replaces an old ventilating shaft. Its total depth is about 270 ft. The material was water-bearing to a depth of about 188 ft. Congelation was adopted for this portion of the work. The contract was given to Gebhardt and Koenig, of Nordhausen. The contractors agreed, barring certain unavoidable accidents, to complete the 188 ft. in fifteen months, from July 1st, 1899. The proprietors of the mine guaranteed that no warm springs or salt-water deposits would be encountered. The contractors were to maintain the ice wall until the lining was placed and a safe connection made with the solid material.

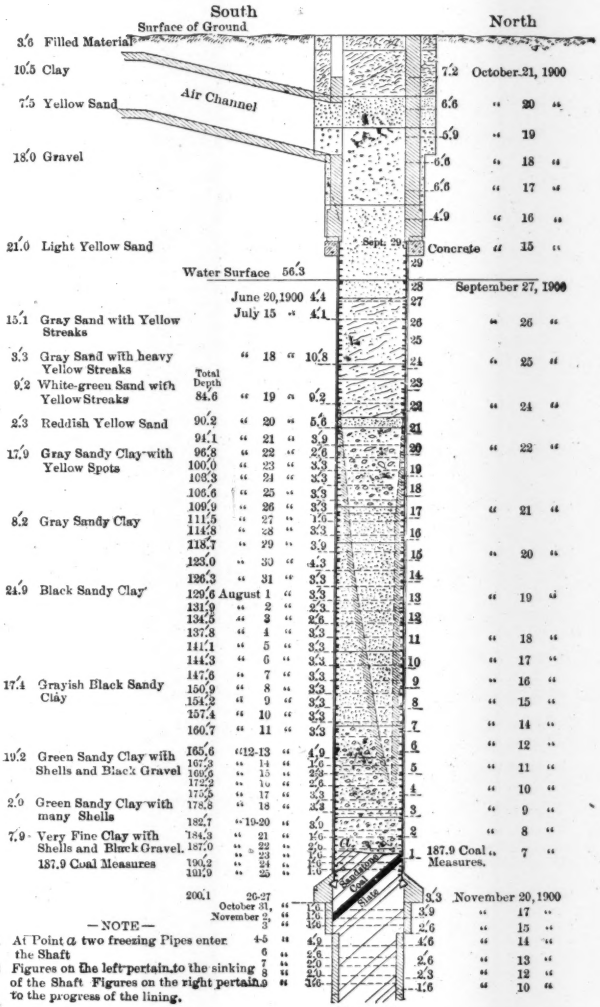
The shaft was circular in plan, and 13.1 ft. in diameter.

The material in the first 188 ft. was clay, gravel, sand, sandy clay and firm clay. The order of their occurrence is shown in Fig. 37. The water level was 56.3 ft. below the surface.

The borings for the freezing tubes were started from the bottom of a well 27.5 ft. deep. There were twenty-four freezing tubes, spaced equidistant on the periphery of a circle 24.6 ft. in diameter. The bore-holes were $7\frac{3}{4}$ ins. in diameter at the surface, and, approximately, 6 ins. at the bottom. They were 190.2 ft. deep. The inside diameter of the freezing tubes was $3\frac{3}{4}$ ins. The inside diameter of the circulating tubes was 1 in., and the metal was $\frac{1}{4}$ in. thick. The freezing tubes were provided with elastic joints, in order to permit their

SECTION OF RECENT FORMATION PASSED THROUGH FOR AIR SHAFT FOR MINE MARIA

From *Gluckauf*, 1901.



June 20, 1900	4.4				
July 15 "	4.1				
" 18 "	10.8				
84.6	" 19 "	9.2			
90.2	" 20 "	5.6			
94.1	" 21 "	3.9			
96.8	" 22 "	2.6			
100.0	" 23 "	3.3			
103.3	" 24 "	3.3			
106.6	" 25 "	3.3			
109.9	" 26 "	3.3			
111.5	" 27 "	1.6			
114.8	" 28 "	3.3			
118.7	" 29 "	3.9			
123.0	" 30 "	4.3			
126.3	" 31 "	3.3			
129.6	August 1 "	3.3			
131.9	" 2 "	2.3			
134.5	" 3 "	2.6			
137.8	" 4 "	3.3			
141.1	" 5 "	3.3			
144.3	" 6 "	3.3			
147.6	" 7 "	3.3			
150.9	" 8 "	3.3			
154.2	" 9 "	3.3			
157.4	" 10 "	3.3			
160.7	" 11 "	3.3			
165.6	" 12-13 "	4.9			
167.3	" 14 "	1.6			
169.6	" 15 "	2.3			
172.2	" 16 "	2.6			
175.5	" 17 "	3.3			
178.8	" 18 "	3.3			
182.7	" 19-20 "	3.9			
184.3	" 21 "	1.6			
187.0	" 22 "	2.6			
"	" 23 "	1.6			
190.2	" 24 "	3.2			
191.9	" 25 "	1.6			
200.1	26-27				
	October 31, "			3.3	November 20, 1900
	November 2, "			3.9	" 17 "
	3 "			2.6	" 15 "
	4-5 "			4.6	" 14 "
	6 "			2.6	" 13 "
	7 "			2.6	" 12 "
	8 "			2.3	" 12 "
	9 "			1.6	" 10 "

— NOTE —

1 At Point *a* two freezing Pipes enter the Shaft

2 Figures on the left pertain to the sinking of the Shaft Figures on the right pertain to the progress of the lining.

FIG. 37

contraction under low temperatures. The casings of the bore-holes were withdrawn after the freezing tubes were placed. The collecting and circulating rings were placed at the bottom of the well.

The freezing apparatus was driven by a 120-H.-P. engine. The freezing medium was chloride of magnesium. The freezing machine was started on March 23d, 1900, but, owing to delays for some improvements made in the apparatus, it did not work regularly until May 25th.

During the delay, the shaft was excavated down to ground-water level and lined with **U**-rings of fluted tubbing. On June 20th, 1900, the excavation was carried to a depth of 4.4 ft. below water level. The material was yellow sand, and water entered from the sides, exposing some of the freezing tubes.

Excavation was discontinued, and the shaft was refilled with sand and clay. On July 16th, excavation was resumed, although the unfrozen core was still 14 ft. in diameter. The size of the core decreased gradually, and, at a depth of 33 ft., it became necessary to point the walls of the shaft. Later, resort was had to blasting with compressed powder. The holes were about 4 ft. deep, and inclined at an angle of 75° toward the center of the shaft. The charges used consisted of from 7 to 10 oz. of powder. In sinking the shaft, there were three 8-hour shifts of eight men each per day. The 131 ft. below ground-water were excavated in forty days. It was feared that the low temperature would have an injurious effect on masonry. Iron tubbing was adopted for the lining.

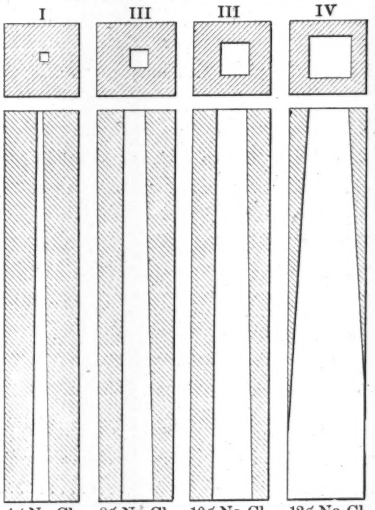
As shown in Fig. 37, the soft core was found to be located quite eccentrically to the center of the shaft. Two of the bore-holes had deviated so far from the perpendicular that they were encountered by the excavators. The tubes were thrown out of commission, temporarily, and those portions within the shaft were cut off. The end was then closed and the use of the shortened tubes resumed.

A leak occurred in the ice wall a short distance below water level, but the water was collected in a pipe and hoisted to the surface.

Ronnenburg Alkali Works.—These works are near Hannover, in Germany. Work was begun in 1898 and completed in 1901.

The first plan adopted was to sink by ordinary methods for 131 ft., and then, with hydraulic pumps, force down a cast-iron shell pro-

From *Gluckauf*, 1901
To Accompany Description of Ronnenberg Mine



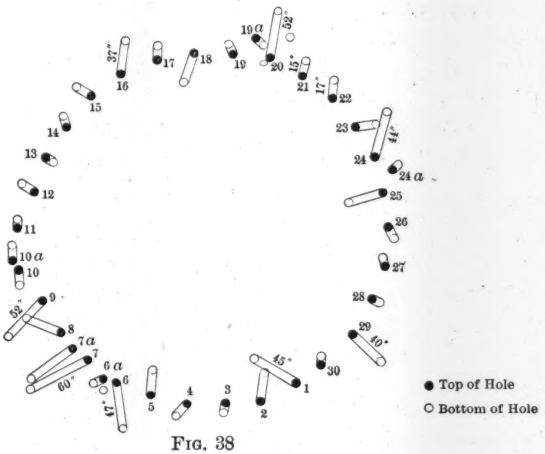


FIG. 38

vided with a cutting edge. The inside diameter of the first section was 21.3 ft. It was lined with masonry 2.6 ft. thick. Below a depth of 23 ft. the quantity of water pumped was 24 cu. ft. per second, and, below a depth of 88.6 ft., the character of the material proved to be very bad. At a depth of 105 ft. the masonry walls were displaced laterally 4 ins. In spite of all effort, the pit soon filled with water up to a depth of 25 ft. After experiments had shown that the ground-water, which contained about 3% of salt, could be frozen, the Poetsch process was adopted. Tests were made with brine solutions containing, respectively, 4, 8, 10, and 12% of chloride of sodium. These solutions were placed in the ice machine and submitted to a temperature of —12° Cent. for 48 hours. The shaded areas, in Fig. 38, show the ice formed in that time. The proportions of salt in the ice and in the uncongealed liquid are also given for each case.

The finished diameter of the shaft was 19.7 ft.

The material penetrated was as follows:

Soil	1.6 ft.	Firmly compact gypsum	128.8 ft.
Yellow sandy clay	14.8 "	Very firmly compact gypsum	18.0 "
Sand and clay	10.7 "	Gypsum, with clay	59.7 "
Sand	2.3 "	Gypsum, with gray and green clay	4.8 "
Blue and yellow clay	4.7 "	Green clay, with grains of quartz and apatite	41.0 "
Blue clay	51.2 "	Firmly compact gypsum.	17.5 "
Gypsum	30.7 "	Gypsum, with clay	0.5 "
Fissured gypsum	24.6 "	Firmly compact gypsum.	32.8 "
Gypsum	2.8 "	Total	459.3 ft.
Sand	2.6 "		
Gypsum	5 9 "		
Fissured gypsum	4.3 "		

The freezing tubes were put down on the periphery of a circle having a diameter of 29½ ft. Considerable trouble was caused by the deviation of the holes from the vertical. In all, thirty-five holes were bored, but probably only thirty were used for congelation. The freezing tubes were 413 ft. long. One hole was bored in the center of the shaft to a depth of 2 953 ft., but, afterward, it was plugged at a depth of 577.5 ft. The diameter of the freezing pipes was 5 ins. and the thickness of the metal ¼ in. Common gas pipe, 1½ ins. in diameter, was used for the circulating pipes. The circulating and collector

rings were of cast-iron pipe, 11 ins. in diameter. The pipes above ground were painted with a coating of tar and cork and covered with cork and ashes. The rings were placed at a depth of about 6 ft. below the surface, to avoid the effects of surface temperature as far as possible.

In th design of the freezing apparatus, its capacity was doubled on account of the salt in the ground-water. The calculated capacity was 240 000 frigories per hour at —25°, or 300 000 at —15° Centigrade. A test, however, showed the actual output to be 383 000 frigories at —15° Centigrade.

The refrigerator was of the Fixary type, and was driven by two engines developing 70 or 80 H.-P. A 28% solution of calcium chloride was used in freezing.

Congelation was started on January 19th, and excavation about May 1st, 1901. At that time the temperature of the solution, on returning to the machine, was about the same as when it entered the pipes, *i.e.*, —20° Centigrade. On July 15th, a depth of 226 ft. had been reached without incident. Blasting was permitted only in the center of the shaft, and excavation was carried on mainly with picks and shovels.

Pit No 2, Ligny-Les-Aire.—This shaft is near Estrée-Blanche, in Northern France, and was sunk in 1900–1901.

The material penetrated was as follows:

Vegetable earth	7.4 ft.	Yellowish marly chalk..	74.6 ft.
White marly chalk.....	59.1 "	Bluish clayey chalk.....	154 2 "
		Total.....	295.3 ft.

The water level was at a depth of 114.8 ft., and there was a great deal of water to contend with.

The finished diameter of the shaft was 13.1 ft. Before starting the borings, a pit, 23 ft. in diameter, was sunk to a depth of 9.8 ft. Eighteen holes were then put down from the bottom, on the circumference of a circle having a diameter of 19.7 ft. The depth of the holes was 305.1 ft. The borings were commenced on May 1st and completed on September 29th, 1900.

A Fixary machine, having a capacity of 2 200 lbs. per hour, was used for congelation.

The period of excavation and freezing extended from November

9th, 1900, to June 24th, 1901. The total time required was two hundred and twenty-seven days.

Washington Colliery.—This colliery is near Washington, in County Durham, England.

The first or larger shaft to which this description is mainly devoted was sunk in 1902.

Some old abandoned openings existed on the property, but the owners decided to install a modern system of working, and discarded the old pits. The adopted plan called for two circular shafts, 14 ft. and 12 ft. in diameter, respectively. As there had been much trouble in sinking by ordinary means, in the same locality, the Poetsch process was adopted from the first. Considerable interest is attached to this sinking, as it is said by Mr. Mark Ford, the manager, to be the first application of the process in England. The contract for both shafts was given to Messrs. Gebhardt and Koenig, of Nordhausen, Germany. The installation of much of the plant was made with regard to the sinking of the smaller shaft, but, from the information at hand, it does not appear that actual operation were begun on the smaller until after completion of the larger pit.

The material penetrated was as follows:

Soil	1.3 ft.	Clay, with boulders, dry.	12.9 ft.
Yellow sand, dry	34.5 "	Loamy clay, dry	5.2 "
Gray sand, wet	41.3 "	Stiff clay, with boulders, dry	9.6 "
Blue clay	0.1 "	Yellow freestone	13.0 "
Gray sand, with a gravel bed, damp	2.3 "	Total	120.2 ft.

The wet sand contained about 19.6% of water.

As a preliminary to sinking the tubes, a well, 24 ft. in diameter, was sunk to a depth of 24 ft. This well was lined with plank supported by wooden cribs, 6 ins. square and spaced 3 ft. apart. Twenty-two bore-holes were spaced equidistant around a circle having a radius of 10.25 ft.

The borings penetrated the freestone 1 ft. 6 ins. The holes were 6 ins. in diameter, and considerable trouble was experienced in keeping them vertical, especially through the boulder clay. Four-in. freezing tubes with outside sleeve joints were inserted in the holes, and the casings withdrawn. Six sets of casings served for twenty-

two holes. The circulating tubes were 1 in. in diameter. The circulating and collector rings were about 8 ft. above the bottom of the well. In the middle of the shaft was placed a tube, 18 ft. deep, from which the temperature and height of the water were noted, "as the gradual increase of the ice wall slowly caused the water to rise." Before starting congelation, the top of the shaft was enclosed and the exposed pipes were covered with straw rope.

The freezing plant was of the ordinary ammonia type, and was in duplicate. The engines were of 55 H.-P. The circulating medium was a solution of chloride of magnesium containing about 26% of salt. This solution freezes at a temperature of $-34°$ Centigrade. The temperature of the solution on leaving and returning to the refrigerator is shown clearly in Fig. 10. The brine was circulated at the rate of 144 galls. per minute. The initial freezing period covered forty-three days.

On May 5th, 1902, at the end of the forty-three days, excavation was commenced. The sinking was made 17.8 ft. in diameter to allow for lining. Through the sand, a core, 6 ft. in diameter, was always soft, and the remainder could be worked readily with a pick. In eleven days 55 ft. were excavated. In the clay, however, the ice wall was much thicker. The central core was partially frozen, and never more than 2 ft. in diameter. Recourse was had to blasting, but, owing to the brittle condition of the tubes, great care had to be exercised. Gelignite was used as the explosive, and the charges consisted of $\frac{1}{4}$ lb. to a hole. Brine had to be used instead of water in drilling, as the latter froze so quickly that it held the drills in the holes. One of the freezing tubes was split, probably owing to the effects of a shock, and had to be thrown out of use. In the sand, the shaft was sheathed temporarily with wooden cribs, 6 ins. square, and lagged with planks. For a short distance in the wet sand, iron was used, instead of wood. As soon as the freestone was reached, a cast-iron segmental ring was placed, and from this the permanent lining of brickwork was started..

In general, the lining consisted of two concentric rings of brickwork, each 9 ins. thick, with a 2-in. annular space between them. This space was filled with cement. The brickwork was laid in cement mortar. To prevent the mortar from freezing, it was mixed with water containing 7% of caustic soda. After the completion of the

lining, the freezing tubes were removed and the holes filled with cement mortar. The removal of the tubes was facilitated by the circulation of steam, but, in some cases, the lower sections were lost.

There was some difference of opinion as to the necessity of freezing the boulder clay, but the contractors refused to take any responsibility as to the sinking of the second pit if it were not done.

Shaft No. 1, Laura and Vereeniging Colliery.—This colliery is in the Province of Limburg, Holland.

The date of the work is not given, but was probably some time during the years 1900 to 1902, inclusive.

As about 270 ft. of watery sand were to be penetrated before the coal measures were reached, the Gebhardt and Koenig process was adopted. This, apparently, is the Poetsch process improved and modified in some of its details.

The shaft is 14.8 ft. in clear diameter.

The material penetrated was as follows:

Loam	26.6 ft.	Greensand	4.9 ft.
Gravel	4 6 "	Clay, mixed with shells	8.2 "
Greensand	14.4 "	Sandy clay	13.1 "
Yellow sand	13.1 "	Greensand	36.1 "
Greensand	81.3 "	Gray sand, with brown streaks	13.1 "
Green clay	45.9 "	Greensand	48.5 "
Greensand	6.6 "	Total	323.0 ft.
Green clay	6.6 "		

The water level was at a depth of 27.9 ft. The 295.1 ft. between the water level and the coal measures were all heavily water-bearing. The coal measures, themselves, to a depth of 333.0 ft. below the surface, were fissured, and, consequently, contained considerable water. Below this there was a layer of compact sandstone, then compact, slaty ground. Previous to sinking the freezing tubes, a well, about 30 ft. in diameter and 28 ft. deep, was sunk and lined with rings of channel irons supporting planking.

From the bottom of this well, twenty-four freezing pipes were put down on a circle having a diameter of 23.9 ft. The freezing tubes were 4 ins. in diameter, and were provided, at about mid-depth, with elastic unions to prevent breakage by contraction. The depth to which the tubes were placed is not stated, but, probably, they did not pene-

trate very deeply into the coal measures. They were tested carefully as to their verticality, and, later, were found to be practically true. The circulating tubes were 1¼ ins. in diameter. The distance between the borings was about 2.9 ft.

The cold-producing apparatus was of the carbonic-acid type, and a double compressor was used. The refrigerating liquid was a 30% solution of chloride of magnesium, and left the cooling tanks at a temperature of —20° Centigrade. In eight days after the freezing machine was started, the wall of ice around each of the pipes was 20 ins. thick. During the early portion of the freezing period there were several interruptions.

Sinking was begun three months after congelation was started. For the entire distance down to the coal measures there was an unfrozen central core. Excavation was carried on with hammers and chisels. No powder was used, as there was some fear of injuring the ice walls. There were occasional soft spots, due to the escape of the freezing solution, but these did not cause any trouble. The rate of progress was about 4.9 ft. per day, and the coal measures were reached without incident. However, on attempting to widen the shaft, in the coal measures, a collapse occurred on the south side of the shaft. The pit was partially filled with water, and sand rose to a height of 131 ft. In order to prevent further damage to the ice wall, the shaft was filled with water up to its natural level. To strengthen the ice wall, six tubes were put down on the south side of the shaft and on the periphery of a circle 36 ft. in diameter. Twelve tubes were also sunk inside the shaft as near as possible to the walls. All these tubes extended 33 ft. into the coal measures. In boring the outer holes, no cavities were discovered behind the ice wall, but the ground, evidently, had been greatly disturbed. During this operation, two subsidences of the earth occurred on the south side of the shaft. The sinking of the holes inside the shaft was accomplished with considerable difficulty, particularly as the workmen, on abandoning the shaft, had left all their tools behind. After freezing had advanced sufficiently in the new tubes, the work of excavating the sand was started, and, at a depth of 230 ft., the mass was found to be entirely frozen.

The provisional timbering was uninjured, and the channel rings were in their original position. This indicated that the ice wall had

remained undisturbed. It was soon found that the tubes inside the shaft, particularly those on the north side, had deviated very considerably from the perpendicular. The position of all the holes of the second set, at the level of 278.9 ft. below the ground, is shown in Fig. 39. At the depth of 308 ft. the deviations had increased considerably, and it was not deemed safe to proceed further with the excavation until additional tubes were placed. The bent tubes were removed, and a third series, of fourteen holes, was started on the level of 308 ft. below the surface of the ground, and located as shown in Fig. 40. In order to avoid interfering with the wedging cribs, used for supporting the permanent lining, these new holes were started on an inclination of 5% outward from the vertical. To guard against a sudden inrush of water, the boring rods were worked through stuffing boxes in hollow guides set in concrete at the bottom of the shaft. The cold was severe, the temperature being at times as low as —13° Centigrade. The water for washing out the borings froze quickly even when warmed, and it was found necessary to use a weak solution of chloride of magnesium.

In making these borings, the limit of congelation, indicated by the presence of water, was noted, and is shown graphically on the developed surface, Fig. 42. The freezing tubes had to be inserted quickly in order to let as little water as possible into the shaft. Tube No. 11 gave considerable trouble, as a spring of water broke out along the outside and could not be stopped even after freezing was started. After a number of expedients had failed, it was finally choked, at the end of about four weeks' time. This third series was connected to circulating and collector rings placed about 20 ft. above the bottom of the shaft. After congelation had been completed, Holes Nos. 15, 16, and 17, Fig. 42, were driven to a depth of 349 ft., and showed that the ground was completely frozen. The freezing tubes which were working at this stage are shown in Fig. 41. Forty freezing tubes were now in operation, and it was feared that, through the intense cold, the expansion of the material might fracture the frost wall, and for this reason some of the pipes were put out of use occasionally.

Soon after the resumption of the excavation, it was found that the tubes of the third series were all nearly vertical and did not leave room for placing the ordinary wedge cribs for supporting the lining. At a

From *The Iron and Coal Trades Review*, Vol. LXVI.
To accompany Description of Laura and Vereeniging Shaft.

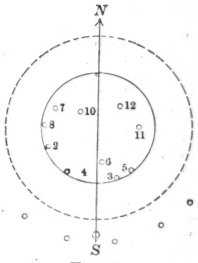


FIG. 39.

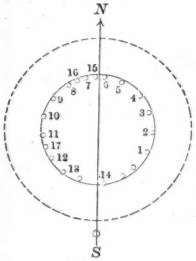


FIG. 40

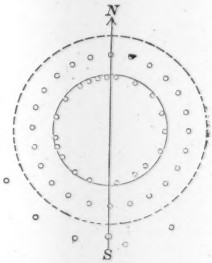


FIG. 41

DIAGRAMS SHOWING LOCATIONS OF BORE-HOLES.

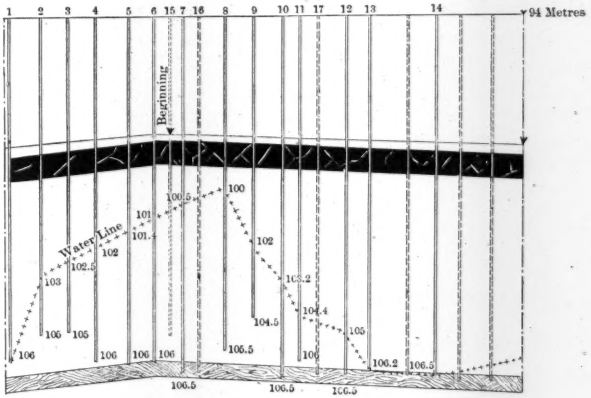


DIAGRAM SHOWING DEVELOPMENT OF BORE-HOLES MARKED IN FIG. 40.
The beginning of the Coal Measures at a depth of 98¼ metres
is shown by the black arrow head.

FIG. 42

depth of 336 ft., excavation was suspended. Two fluted tubbing rings were designed, to take the place of the wedge cribs, and served the purpose admirably. These rings were composed of segments, 60 ins. high, and had two horizontal ribs as well as flanges for the full length of the segment. Between these ribs the web was arched outward. These rings did not differ from the ordinary rings used for the entire shaft, except in the arched webs. The two rings were placed carefully at the bottom of the excavation, and the space between them and the ice walls was filled with a quick-setting concrete. To render the concrete frost-proof, a 20% solution of calcidum was added, and proved very satisfactory. Previous experiments proved that a solution of soda failed in its effect, and that a solution of magnesium chloride diminished the solidity of the concrete. After the tubbing was completed to the surface, excavation was resumed.

The time required for completing the shaft to the coal measures was twenty and one-half months, divided as follows:

Making borings and fitting up tubes	3	months.
Mounting ice machine, etc.	2	"
Congelation	3	"
Sinking, before accident	3	"
Placing additional tubes and congelation	8½	"
Excavation, etc.	1	"
Total	20½	months.

Work is now in progress on a second shaft, and, profiting by experience, the tubes will be sunk about 15 ft. into the coal measures.

Other Projects.

Under this heading are included the projects for and actual application of the freezing process of which only brief mention has been found. Such information as is available is given in each case.

Bucharest, Roumania.—Engineering News, of July 5th, 1884, states that Mr. Poetsch has just closed a contract to sink a series of bridge piers at the above locality. The writer has been unable to find any further details in regard to this matter, and is uncertain whether or not the contract was ever carried out.

Wyoming, Pennsylvania.—Edward L. Abbott, Assoc. Am. Soc. C. E., and a number of the technical journals, refer to the fact that freezing pipes were being placed for sinking a shaft in the Wyoming

Valley, Pennsylvania. No further published information could be found, but inquiry of Mr. R. V. Morris, Chief Engineer of the Coal Companies, Wilkesbarre, Pennsylvania, elicited the following information:

"I find on investigation that the freezing process has never been actually used in shaft sinking in this region, though it has been attempted in one case of which the following is the history given by Mr. J. U. Crawford, now of the Peoples Coal Company, Scranton, Pa.

"In 1889 the Mt. Lookout Coal Company, of which Mr. Crawford was then Superintendent, started to sink two shafts near Wyoming, the strata, as developed by test holes, showing approximately 32 ft. of dry gravel, and 70 ft. quicksand above the rock. The first shaft was sunk by pneumatic process with a square caisson, and, while it was in process of sinking, an attempt was made to sink a second by the freezing process, pipes being put in from 5 to 7 ft. apart in a circle around the proposed shaft driven 102½ ft. to rock and 5 ft. further into the rock itself, which, from the bore-hole indications, was supposed to be solid. A freezing mixture, at a temperature varying from 5° to 7° below zero, was circulated in the pipes for a period of about seven weeks, when the first shaft's caisson reached rock which, instead of being solid, was found fissured for 18 ft. below its surface, and with a large flow of water from the fissures.

"With this information, the Mt. Lookout Coal Company, decided that it would be impossible to succeed with the freezing process, with pipes extending only 5 ft. into the rock, so abandoned the attempt and successfully sunk the second shaft with a pneumatic caisson, finding rock at 102½ ft., as above, but only succeeding in making a water-tight joint in solid rock at a depth of 140 ft. below the surface, the fissured rock extending in this shaft for about 35 ft. below the rock surface."

"While considerable time elapsed between the cessation of the freezing and the sinking of the caisson, the ground passed through is said to have been quite well frozen, and Mr. Crawford gives as his opinion that had it not been for the fissured rock the freezing process would have been a success."

Sulphur Mine, Calcasieu Parish, Louisiana.—For many years attempts have been made by various companies to sink a shaft to the rich sulphur deposits occurring in this parish, but all have failed.

In 1886 the National Sulphur Company proposed to use the Poetsch process. It appears, in the following quotation from a recent letter of F. M. Kerr, M. Am. Soc. C. E., Assistant State Engineer of Louisiana, that no actual work was done.

"No plant for operating the process was ever installed at the Sulphur Mines, in Calcasieu Parish. Only the right to operate was

acquired by myself and associates at the time, and nothing more than an examination into its merits as applied to that locality looked into, as explained in the article to which you refer.

"The conclusion was at the time arrived at that the process was eminently fitted to accomplish the objects in view, but a failure to capitalize the enterprise finally caused us to abandon the project and the process was not put into test there."

The article mentioned in the quotation is one by Mr. Kerr, in the *Journal of the Association of Engineering Societies*, Vol. XXVIII, 1902, page 90. It contains a very interesting account of the various attempts to mine sulphur in this locality.

The Scientific American, in Vol. 73, page 388, and again in Vol. 75, page 280, states that the refrigerating plant was installed, but that congelation was not successful.

Harlem Speedway.—The Engineer, of London, June 28th, 1885, page 556, states that Sooysmith and Company are attempting to freeze a bed of quicksand to be penetrated by the excavation for a retaining wall. The depth of the excavation is about 40 ft. A row of 4-in. pipes is sunk, the pipes being 3 ft. apart. Inside the 4-in. pipes, smaller pipes are placed, through which compressed air (the freezing medium) is conveyed to the bottom of the large pipe. No other information, to indicate the success or failure of the project, has been found.

Lift at Fontinette.—In an abstract in the *Minutes of Proceedings* of the Institution of Civil Engineers, Vol. CXXIV, 1896, page 499, the adoption of the Poetsch process for the repair of the cylinder pits of the canal lift at Fontinette is noted.

The *Journal* of the Iron and Steel Institute, No. 1, 1897, Vol. LI, states that the process was applied successfully.

Shaft near Eygelshoven, Province of Limburg, Belgium.—Brief abstracts from *De Ingenieur*, published in Rotterdam, state that the freezing process had been adopted in several instances for sinking shafts in the vicinity mentioned. The article, apparently, is quite complete, and might afford considerable information if translated. The date of the article is 1901.

Shaft of the Ascot Deep Leads Company near Ballarat, Australia.—Mr. F. D. Johnson * states that:

"Some twenty years ago the diamond drill in passing through the basalt into 'drift' found the hydrostatic pressure sufficient to eject

* *The Mining Journal*, Oct. 25th, 1902.

water and sand above its derrick, or about 450 ft. This pressure has been considerably reduced by pumping in other shafts miles distant. The above-named Company is, for the first time in Australia, trying to get through a not very thick layer of drift below the basalt by employing the freezing process or Poetsch method. At the date of writing there seems to be a little doubt as to its complete success, owing, it is said, to the great pressure of water still present there."

Brussels Railway.—Mr. A. Gobert * states:

"Very many sinkings by the freezing process are contemplated in Belgium, and it was expected that the soil of Brussels would be frozen for the line of railway between the Nord and Midi Railway Stations. The Government had consulted the writer and he had given them a complete report upon the subject."

Empelde, Germany.—The Colliery Guardian of August 7th, 1903, page 314, incidentally, mentions the use of the congelation process by the Schalken, Gruben und Hüttenverein at Empelde, near Hanover, in 1899. No particulars have been found.

Pennsylvania Railroad Tunnel Under the Hudson River.—From time to time it has been suggested that the freezing process could be used to advantage in constructing tunnels under the Hudson River. Since the Pennsylvania Railroad tunnels have been projected, plans worked out in some detail have been published in the technical papers.

Shafts Sunk by Gebhardt and Koenig.—In the advertisements of this firm they claim to have completed successfully the following shafts:

Two shafts for the Société Anonyme des Charbonnages, Willem Sophia, in Spekholzheide (Limburg, Holland).

One shaft for the Vereinigungs Gesellschaft für Steinkohlenbau, in Wurmrevier, zu Kohlscheid.

One shaft for the Gewerkschaft Braunkohlenbergwerk, Consol Sophie, near Wolmirseben.

One shaft near Gusten für die Herzogl. Anhalt Regierung.

Two shafts, 5 m. diameter, for the Consol Alkali Works at Westeregeln.

One shaft for the Con. Braunkohlen Mine, near Atzendorf.

One shaft in Essen, on the Ruhr.

One shaft, 147 m. deep, 6.1 m. diameter, for the plant Auguste Victoria, at Recklingshausen.

No further particulars are available.

* *Transactions*, North of England Institute of Mining and Mechanical Engineers, Vol. LII.

Authorities.—The data in this paper have been gathered from the following publications:

Allgemeine Bauzeitung.
Annales des Mines.
Berg und Hüttenmännische Zeitung.
Bulletin de la Société de l'Industrie Minérale.
Colliery Guardian.
Engineering (London).
Engineering and Mining Journal.
Engineering News.
Glückauf.
Iron and Coal Trades Review.
Journal of the Association of Engineering Societies.
Journal of the Iron and Steel Institute.
Journal of the Western Society of Engineers.
Le Génie Civil.
Mining Journal.
Minutes of Proceedings of the Institution of Civil Engineers.
Oesterreichische Zeitschrift für Berg und Hüttenwesen.
Proceedings of the Institution of Mechanical Engineers.
Proceedings of the South Wales Institute of Engineers.
Railroad Gazette.
School of Mines Quarterly.
Scientific American Supplement.
The Engineer (London).
The Engineering Record.
Transactions of the Federated Institute of Mining Engineers.
Transactions of the Mining Institute of Scotland.
Transactions of the North of England Institute of Mining and Mechanical Engineers.
Zeitschrift des Vereines Deutscher Ingenieure.

A list of these authorities, with details as to year, volume, page, etc., for each mine, shaft, or other work described, is filed in the Library of the Society, where it may be examined by anyone who is interested in the subject.

DISCUSSION.

GEORGE E. THOMAS, M. AM. SOC. C. E. (by letter).—The writer has read this paper with much interest. Mr. Thomas.

The ground pipes for the shaft at the Chapin Mines, Iron Mountain, Mich., were put down by the Chapin Mine Company, and have been described fully by D. E. Moran, M. Am. Soc. C. E., in a paper* before the School of Mines of Columbia College, New York City. This piping was finished in the summer of 1888, and, in August of that year, the writer was ordered by General William Sooy-Smith, President of the Poetsch, Sooy-Smith Company, to take charge of the work.

The design of the circulating piping and the whole installation of this plant was one of the writer's most interesting experiences. There were twenty-six stand-pipes, each with a specially designed head, the 8-in. cap being a special casting; also the connection above the 4-in. nipple; and every joint was soldered, to make certain that there would be no leakage. The 1½-in. intake pipe was extended down to 10 ins. above the bottom of the 8-in. stand-pipe, thus giving ample space for inflow.

The refrigerating machine was started on November 19th, 1888. In twenty-four hours all the piping above ground and the circulating pumps were coated with frost. Previous to starting the freezing, the writer had put down a system of test pipes to afford information as to the distance the freezing had extended from the pipes.

The explosives used were lime, black powder, and then dynamite. Large quantities were thrown out by each blast, the frozen material being so strong that the charge would split boulders instead of yielding before them.

The frozen sides of the shaft were also very firm. A depth of 40 ft. was excavated before any timbering was put in, and, at a depth of 80 ft. below the surface, a space of 7 ft. was left untimbered.

The leak which developed was caused by a boulder on the rock under one of the stand-pipes, and this proved to be a valuable experience. The temperature of the water coming into the shaft under this boulder was +45° Fahrenheit.

The adjustment of the circulating fluid, to harmonize the temperatures in the twenty-six stand-pipes, was one of the difficulties, and was overcome by the use of valves on the stand-pipes. There was not a variation of 0.1° in the temperature of any of the stand-pipes. Recording thermometers were used and gave good results.

Tests of the frozen material gave the following results:

Average of four tensile tests	= 431 lbs. per square inch.
Maximum	= 491 " " " "
Average of eight compression tests	= 575 " " " "
Maximum	= 880 " " " "

* *School of Mines Quarterly*, Vol. XI, p. 237.

Mr. Thomas. From August, 1888, when this work was started, the writer was alone, and working continuously, night and day. Mr. Moran came to assist on January 14th, 1889, and the work was finished early in the spring of that year. It was the writer's purpose to have written a paper on this subject, but other work prevented. He has a perfect history of the work and a number of photographs; also specimens of some of the finer material through which the shaft passed. One boulder weighed more than 17 tons.

This system of holding flowing material, such as silt, quicksand, or any form of unstable subsoil, is absolutely safe and reliable. Experience has demonstrated that refrigeration can safely be stopped for hours, or even days, after the frozen material has reached a consistency which is understood to be strong enough to withstand the required pressure. In comparison with the pneumatic process, the fact that there is no excessive pressure of air on the workmen is in its favor. The temperature at the bottom of the shaft at Iron Mountain was —25° Fahr., and was not any discomfort to the men when working. Such work, however, must be looked after carefully from start to finish, there being a law of compensation in all things.

Mr. Moran. D. E. MORAN, M. AM. SOC. C. E.—This paper is valuable in that it gives a *résumé* of the literature on the freezing process. In the opinion of the speaker, it is not an engineering paper on the subject of the freezing method; but is a *résumé* of the literature. It is very modest, and the author has made no claims for it beyond the fact that it is a condensation of what he could find printed on the subject. The speaker would like to see it treated on a more ambitious scale, taking up the theory, the practice and the results accomplished.

The entire subject seems to be of interest to every engineer. It is a new field, in America at least, and the problems presented are interesting and complicated, as they involve thermodynamics and statics. Civil, mechanical and mining engineers are all interested.

It is to be regretted that no one has thought it worth while to investigate the freezing process, and give answers to many of the questions, which, so far, seem to be unanswered. Some of these involve the higher mathematics, and the speaker has tried to solve them, but without success. It is desirable to have answers to certain questions which must arise wherever this process is to be used.

The only work done by the freezing method in America, as far as the speaker is aware, has been in charge of civil engineers, who are members of this Society, and probably most of them live in New York. The author refers to one or two articles written by members of this Society, and notes minor discrepancies therein. These discrepancies could have been explained by any one of the four men who were on the works. In the case of one alleged failure which he records, the author took the trouble to inquire and write letters to

ascertain the difficulty. To the best of the speaker's knowledge and belief, the author did not ask any one of the four members of this Society who knew the real reasons why there was a failure or a partial failure of that work; but, he wrote elsewhere, and received a reply from a man who is not a civil engineer nor a member of this Society, the result being, that, while much of the information obtained was true, it was not an engineering answer to his question and it leaves much to be explained. Mr Moran.

These criticisms are made because this subject is of great interest. They are made with all the more earnestness, because, while the author has not expressed his own opinions nor recited a single experience, nor advanced a single theory of his own, the speaker believes that he has ideas, understands the theory of the subject, and has had some experiences to record, and therefore would call on him for his own ideas, experiences and theories, rather than the best possible *résumé* of printed literature, which is available to all.

The author is to be thanked for having presented this paper, because it gives those who do not care to make search a chance to read an interesting paper and get the literature on the subject. But it would have been better if the author had considered the problem from the standpoint of scientific engineering, and had given something more than a record of failures and successes in foreign countries, which failures or successes are not explained, and, as to which, he frankly confesses, some of the most important and interesting points can only be left to guesswork.

The speaker is a firm believer in the freezing process. The problems relate to the conductivity of materials, the specific heat of the materials to be frozen, the so-called latent heat of the water contained in the material, the rate of transmission of heat through material, and the effect of varying the temperature of the brine relatively to the temperature of the material involved.

It has been stated that the size of the ice machine required could be determined if the specific heat of the material to be frozen and the percentage of water contained in that material were known; but something must be added, because an ice machine has to do something beyond the zone of freezing. This represents a loss, and is one of the interesting questions to be settled before any calculation can be made as to the exact number of refrigerating units. These are only a few of the interesting questions of theory. There are also interesting questions of detail. One was touched upon in the paper, and related to the rupture of the pipes. At Iron Mountain, as described by Mr. Thomas, 10-in. pipes were sunk to the rock as pilot pipes, and 8-in. pipes, sealed at the bottom, were then inserted inside of the pilot pipes. The exterior 10-in. pipes were then withdrawn, leaving the 8-in. pipes in contact with the soil. The circulating pipe was then

Mr. Moran. put inside the 8-in. pipe. There was no trouble with any of these pipes. At Wyoming the same process was followed, up to a certain point. The 10-in. pilot pipes were driven and smaller 8-in. pipes were placed inside of them, and then an attempt was made to withdraw the exterior pipe. In a number of cases it failed to come out, and so the circulating pipe was put in and the ice-machine started, leaving a steel pipe outside of the freezing pipe. Thus an annular space full of water was left between the two pipes. The first effect of the freezing was to freeze a thin film of ice on the exterior of the inner pipe. The inner pipe had sleeve couplings; the exterior pipe was flush at the joints, both inside and outside. The conductivity through the sleeve couplings was the same as the conductivity through the pipe and the thickness of ice was the same. The result was that the frozen wall of water extending out from the inner pipe first met the exterior pipe opposite these sleeve couplings, which were 20 ft. apart. The interesting experiment was thus had of a certain quantity of water freezing in a confined space. There were an interior and an exterior cylinder, and an annular space filled with the confined water. As soon as the water commenced to freeze it expanded. It might have been expected that the expansion would burst the exterior cylinder; instead of that, it sheared the 8-in. pipe under the coupling, so that, when the pipe was withdrawn, long afterward, it came up in pieces, and it was seen that the thread had been sheared. It was a case of shear, not a case of tension; and the explanation is very simple.

That brings up another practical question relating to the ground pipes and the whole system. What becomes of the water? A case is mentioned in the paper where it was expected that this same experiment would be repeated, but on a larger scale, by freezing so as to confine the water. When the water freezes, what will happen? The speaker thinks there is nothing to be apprehended on that score. There might be a slight relative motion of the pipes in the ground, but as a block of ice under pressure in a glacier or elsewhere can be broken and will yet freeze again under pressure and at a suitable temperature, so the speaker believes a similar action would have taken place here, and therefore no trouble need have been apprehended from that source. If the rupture occurred, it could heal itself. That is one of the beauties of the freezing process.

The Wyoming shaft was finished by the pneumatic process. The story is told of an old professor who, when an experiment failed, said: "Gentlemen, this experiment has not failed. No experiment can fail." In that sense, the experiment at Wyoming did not fail. The material was frozen. The work was done by Mr. Sooysmith, and the speaker was there occasionally, but had nothing to do with carrying out the work or designing it, any more than he had at Iron Mountain, because he only went to the last-named work as assistant to Mr.

Thomas; while, at Wyoming, he had no position except that of an observer. He can say that the Wyoming experiment taught him a great deal about the freezing process, including that little point about the pipes. Mr. Moran.

The freezing process has its limitations, as has every other system. That the ground was frozen, was proven when the pneumatic caisson was sunk, months afterward, because it was then necessary to go through the frozen ground. The speaker does not wish to describe the various things that happened at that Wyoming shaft. If Mr. Sooysmith had given it his personal attention, the speaker is sure that many things that did happen would not have happened. A freezing job is much like an experiment in a physical laboratory, in that it has to do with things that the ordinary superintendent does not understand. It is an engineering problem and a laboratory experiment on a large scale, and ought to be thus treated. In an article* on the Iron Mountain shaft, written by the speaker, from which some of the illustrations in the paper are reproduced, there is a calculation of the amount of freezing effect required to freeze the sand. In speaking of freezing the sand, what is meant is the reduction of the temperature of the sand, and the amount of freezing effect required to freeze the water in the sand. At Iron Mountain the percentage of water was very low, only 17 per cent. The speaker was surprised to find how much of the freezing effect was required to freeze the water and how little was required to reduce the temperature of the sand, but the proportions are not remembered. A very small fraction of the total effect is required to reduce the temperature of the sand and water. Where there is rock the conditions are reversed, because there is a very small percentage of water in rock, and, therefore, it can be frozen very quickly, as has been shown to be the case in every freezing job. It was shown at Iron Mountain, where the pipes went down through a nest of boulders and the freezing effect had extended to a much greater distance from the pipes than in sand containing only 17% of water. Most of the experiments mentioned in the paper have been with inadequate ice machines. The modern ice machine is capable, if so desired, of giving a temperature of 40 or 50° below zero, and therefore gives very much greater results. It was stated in the paper that the pipes should be 4 ft. apart. One question which it would be desirable to consider is the basis on which the lay-out of the pipes must be designed, with different degrees of temperature in the ice machine. At Iron Mountain, the temperatures were so little below freezing point that on many days but little progress was made.

It has been suggested that all ought to be interested in this question because practically all who have had any outside work know how much more expensive it is to excavate material in freezing than in warm weather. If, when a difficulty is met, that difficulty can be

* *School of Mines Quarterly*, Vol. XI.

Mr. Moran. turned around and made to help the next time, the result is a success. That is what is to be done, some time, with freezing. It is to be made to help. It is expensive to excavate frozen material when frozen material is not wanted, but when it is desired to change a soft, wet, flowing material into something that is as hard as sandstone, a freezing machine should be used to put the material into the desired condition.

Mr. Abbott. E. L. ABBOTT, ASSOC. AM. SOC. C. E.—The speaker has examined the literature on the freezing process as applied to shaft work, and has studied its proposed application to tunneling in wet ground. The records of shaft work are not as complete as could be desired. Although information about the performance of refrigerating machinery is well known or may be easily obtained by test, exact data are required as to what happens below the ground line, in reference to the conductivity of ice and soils, and to what extent their conductivity is increased by a reduction of temperature.

The results of laboratory tests on the conductivity of ice, as made by different physicists, vary greatly, and can be reconciled only by the supposition that they were made at different temperatures or under some other unknown conditions, which would indicate that the law of its conductivity is not well known.

While much practical information has been obtained at various shaft works, it can only be applied properly at other places when the conditions are similar, and where they are not similar it is usual to provide a liberal excess of refrigerating power to cover any unforeseen demand. The cost of refrigeration itself is generally but a small proportion of the total cost of the work.

Thanks are due the author for his compilation of many data which will assist in further investigations, and it is hoped that all who have opportunity for obtaining exact information on the subject will place it on record.

Mr. Thomson. T. KENNARD THOMSON, M. AM. SOC. C. E.—The author refers to the Harlem Speedway without giving any American references. The following is given for the sake of completing the record. Messrs. Steward and McDermott were the contractors for this work, and Mr. McDermott states that, after $8 000 had been expended for labor, in attempting to hold the water back by freezing, they were obliged to abandon this process as a failure for that particular locality. The Mr. Steward of this firm was the late Herbert Steward, Assoc. Am. Soc. C. E., and Mr. McDermott is Mr. Charles McDermott, now of the firm of McMullen and McDermott. He is now in Washington, D. C., and no doubt would be glad to give further information if desired.

It would seem to the speaker that the freezing process should only be attempted where the depths are too great for the use of compressed air, that is, where the water is more than 100 ft. deep, as the compressed-air method is cheaper, quicker and more reliable.

Mr. Thomson.

The author states that the freezing process has been proposed for the Pennsylvania Railroad's Hudson River tunnel. There the material to be penetrated is the finest kind of silt. If the freezing process were used for this work the tunnel lining, of course, would have to be placed against the frozen material, and when this material subsequently thawed out it would leave the tunnel floating practically in a jacket of water. The author refers to a case where soil water containing 3% of salt was frozen by doubling the capacity of the freezing apparatus, and, on the other hand, states that someone in the South has made an unsuccessful attempt to freeze a salt mine. It is an open question as to whether the Hudson River silt does not contain too much salt water to freeze readily.

In most cases quoted by the author it is stated that it was not considered safe to blast, and that the frozen material was chopped out with chisels, picks, etc. Frozen earth is the most difficult material the speaker has ever seen excavated. In removing the remains of an old Rocky Mountain snow slide on the Canadian Pacific Railroad, blasting made almost no impression, as each blast simply blew out a small hole.

It is to be hoped that the gentleman who controls the American patent rights of the Poetsch system will explain why so many of the attempts in this country have been failures, for failures often teach more than successes.

Mr. Boecklin.

WERNER BOECKLIN, ASSOC. M. AM. SOC. C. E.—Through the courtesy of the York Manufacturing Company, builders of refrigerating machinery, the speaker is able to present to the Society data, in the form of a proposal made to him, with a specification, covering a plant suitable for excavating purposes.

The specification, including the principal items, follows:

One 27 H.-P. steam engine of the horizontal type, with a 13½ x 12-in. cylinder, direct-connected to two gas pumps of vertical, single-acting type, with 9 x 12-in. cylinders. This machine is guaranteed to give a duty equal to the melting of 20 tons of ice in 24 hours.

One ammonia condenser, of the atmospheric type, consisting of two coils of 1¼-in. ammonia pipe, each coil to be about twenty-four pipes high and about 20 ft. long.

One ammonia receiver and an oil separator, the former being an air-tight tank to receive the ammonia as it is condensed.

One brine cooler, of double-pipe style, one pipe inside the other, consisting of two coils of 2-in. and 3-in. pipe.

One brine tank of sufficient capacity to hold about one-third of the total quantity of brine in circulation.

One brine pump large enough to force the brine through the system.

One 50 H.-P. horizontal tubular boiler, with a 60-ft. smoke stack 26 ins. in diameter.

Mr. Boecklin.

One feed-water pump, 3 x 2 x 3 ins.

One feed-water heater of 50 H.-P. capacity.

The proposal includes all necessary piping and fittings, twenty-one freezing tubes, 8 ins. in diameter and 103 ft. long, with 1½-in. circulating pipes and circulating and collector rings, a sufficient quantity of ammonia to charge the system and a sufficient quantity of chloride of calcium to make the needed brine of proper density.

The cost of the machinery and accessories, furnished and erected, exclusive of the placing of freezing pipes, will approximate $9 250. This figure will not be more than 5% out of the way.

The foundations for the engine and compressors, the boiler setting and the structure for housing the machinery, are items not included in the foregoing price.

Mr. Campbell.

H. J. CAMPBELL, ESQ.—This paper is of very great interest to the speaker. The author's painstaking care in compiling so carefully the history of this mode of excavating merits much praise, and it will surely be of great value to the engineering profession.

A writer of the history of artificial refrigeration can record but little practical progress antedating the last quarter of a century; consequently, it is apparent from the paper that the engineers were among the first to grasp its utility and apply it as a successful means to accomplish an end, which, before its advent, had well nigh defeated their efforts. It is observed that many of the early attempts to excavate by this process were successful only after much delay, others were only partially successful, and others were total failures. As it was not until the early Eighties that refrigerating systems began to be perfected so as to be relied upon, undoubtedly many of these failures were due to the crude machinery used and the lack of attention to essential details in installing the plants.

It is of the utmost importance, in undertaking excavation by this process, that careful observations be taken to determine the nature of the material, its specific and latent heat, and its conductivity of temperature, as on these data will depend the determination of the best means to be used in applying the refrigeration, and the amount that can safely be relied upon to accomplish the work successfully.

If pipes are to be sunk into the ground, as in excavating a vertical shaft, their spacing and arrangement should be governed by the varying nature of the material on the different sides of the shaft, rather than by spacing them evenly, as has been usual.

If the work is in horizontal shafts or tunnels, it is important that the arrangement be such as to ensure an even freezing entirely around the shaft, which can only be accomplished by bringing the pipes, through which the freezing agent passes, into absolute contact with the material to be frozen.

There is one matter which the speaker considers of importance, if

the quickest success would be attained, namely, that the refrigerating agent be metered to the different series of pipes used, and its return temperature taken, in order to determine the amount of work being done at each point and the progress being made, which, further, serves as a check on the previous examinations. Mr. Campbell.

A perusal of the paper indicates that, even in the latest work, the most modern machinery was not used. Had that been done, much time would have been saved in erection, and the freezing would have been accomplished very much sooner, as the brine could have been carried at a temperature from 8 to 12° lower. Further, the intensifying of the temperatures would have gone far toward ensuring the success of the work, and would have reduced its cost materially.

Up-to-date machinery is now self-contained, quickly erected or taken down, and of such design that when standing idle it will not deteriorate to any great degree—no more than, and possibly not as much as, an ordinary pneumatic plant.

Among the opportunities which have come to the speaker's knowledge where refrigeration could have been used to great advantage and would have saved much time and money may be mentioned the tunnel under the Harlem River for the new Croton Aqueduct.

In order to determine the depth necessary to go below the bed of the river to ensure solid rock excavation for the entire tunnel, frequent borings were made across the river, and showed that at a depth of about 150 ft. below the surface the rock was solid. The tunnel was started at that depth, but, after proceeding for about 300 ft. a pilot drilling developed an 18-in. seam in the rock. This seam was heavily water-bearing, so much so that further excavation on that level was abandoned and a bulkhead was put in to prevent future trouble. Further test borings were made, and it was discovered that this fault in the rock was between those borings made previously, and that, otherwise, the rock was sufficiently solid at this elevation to have been excavated by the ordinary means; but in order to avoid this seam it was necessary to go 155 ft. lower, where the rock proved to be perfectly solid.

As rock is a good conductor of heat and cold it would have been comparatively easy to have frozen this 18 ins. and then proceeded on the level at first determined, thereby saving to the city from $75 000 to $80 000 in first cost and the expense of pumping out this extra depth of shaft whenever it is desired to examine or clean the tunnel. There are other points on the Croton Aqueduct where artificial refrigeration could have been used to great advantage.

This tunnel work and the Speedway are on the same side of the Harlem River. The engineers informed the speaker that they had determined by pilot drilling that this 18 ins. was the only stratum which was too heavily water-bearing to have prevented their excavating

Mr. Campbell. at that elevation, and as the rock has a low specific heat and is a good conductor of heat and cold, and as the water was undoubtedly stagnant, there is no doubt that it could have been easily frozen for excavating.

As to the work on the Speedway, it is understood that the material which it was attempted to freeze was quicksand, which would indicate that the conditions were entirely different from those on the Croton Aqueduct and more uncertain. The speaker knows nothing about these conditions or how the refrigeration was applied, consequently he is unable to state why the work was not successful or whether refrigeration could have been applied so as to have been successful.

The science of artificial refrigeration has advanced to such a point that the engineer should have it in mind as a means of accomplishing difficult excavation which, heretofore, has been done by more crude and expensive means.

Mr. Gobert. A. GOBERT, ESQ. (by letter).—The shaft at Ronnenburg could not be completed by the freezing process, because the ground-water contained too much salt.

In the new coal field of Campine, in the north of Belgium, and in other places on the left side of the Rhine, much salt water is found at

SHAFT SINKING AT A GREATER DEPTH THAN IS POSSIBLE BY THE KIND-CHANDRON PROCESS.

In this case, the soft ground is filled with salt water which prevents the application of the freezing process.
Fresh water is pumped into the place occupied by the salt water.

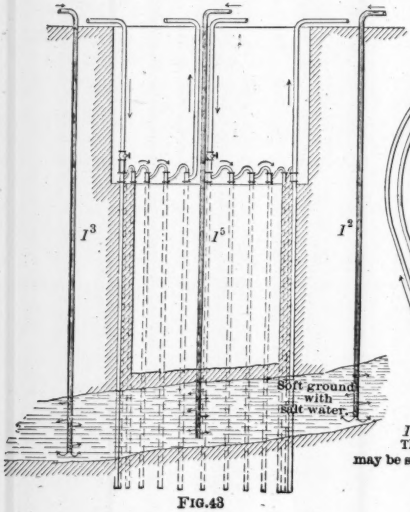


FIG. 43

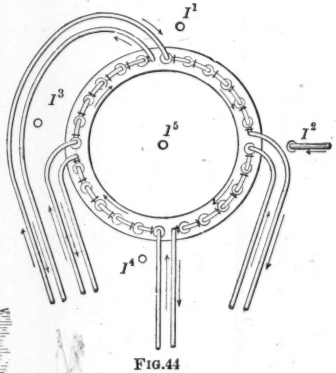


FIG. 44

I^1 I^2 I^3 I^4 I^5 Pipes injecting fresh water. The pipes used for pumping out the salt water may be similar to those used in pumping petroleum.

a depth of, say, 600 m., and is thus beyond the limit, 400 m., ascribed to the possibilities of the Kind-Chandron process. The writer, therefore, proposes to pump the salt water from the soft ground, in the same way as petroleum is pumped, and then pour in fresh water to take its place, thus rendering freezing possible. Mr. Gobert.

This idea is shown clearly in Figs. 43 and 44.

All the writer's recent patents are based on the circulation of brine; he has abandoned the idea of evaporating ammonia in the freezing tubes.

CHARLES SOOYSMITH, M. AM. SOC. C. E. (by letter).—The writer did not have the advantage of being present at the reading of this paper, which presents a general *résumé* of the freezing process as applied to shaft work, but wishes to add his compliments to those already expressed for its usefulness. Mr. Sooysmith.

The record of the experience in sinking shafts by freezing, which makes up most of the paper, is a remarkable tribute to the success of the process when it is considered that its use has been called for only where the difficulties to be overcome have been so considerable that older and simpler methods were insufficient or too costly, and considering that the record is of the early stage of the growth of an entirely new engineering method.

Trouble has occurred in many cases through difficulties or lack of skill or experience in getting the ground pipes into correct position and in preventing leakage. In some cases the freezing plant has had too small a capacity, resulting in insufficient freezing or undue delay and expense. In no case of which the writer is aware has failure been due to any defect or inefficiency in the freezing process itself.

In those cases where in sinking pipes they diverge so as to leave greater distances between them than intended, the result sought can generally be obtained by providing an ice machine of such capacity and kind that the cooling fluid may be circulated at very low temperatures.

The works so far done by the freezing process have, in comparison with large engineering enterprises, been comparatively small, generally requiring such economy in provision of plant that ice machines, of the kind and capacity suitable for producing the low temperatures which should have been used to give the best and most certain results, have seldom been installed. Also, to obtain ease of excavation of the central core, it has generally been thought desirable not to freeze it. These causes have limited the thickness of the frozen wall, inviting likelihood of leaks and weak spots.

At the present time, the development of refrigerating processes has reached such a stage that there is no practical difficulty in circulating the cold fluid at —20° or —30° Fahr., and perhaps lower. There is at least one refrigerating plant in this country in constant operation with a chloride of calcium brine circulation at —30° Fahr.

Mr. Sooysmith. The advantages of low temperatures may be well shown as follows: After the preliminary cooling of the ground is done, the effective head for doing the freezing is the number of degrees of temperature of the cold fluid below the freezing point, and the amount of freezing done in a given time is nearly proportionate to this difference. The total refrigeration of the ground may be divided into two parts: One part to freeze the ground, including its necessary cooling, and the other part to cool the ground outside of that to be frozen. This latter part is loss. But if such a temperature of the circulating fluid be used as would give a temperature head twice as great, a given amount of freezing would be done in approximately half the time, and the loss, meanwhile, would be half as great as in the case of brine having the higher temperature. In the practice described in the works mentioned by the author, a temperature of brine in ground pipes of from 5 to 15° above zero has generally been used, corresponding to an effective head of from 27 to 17 degrees. Had machines been used, furnishing a temperature of —30° Fahr., the effective head would have been some 60° Fahr., or two or three times as great as was used, and the amounts frozen in a given time would have been correspondingly greater. Such temperatures, also, would readily freeze salt water of any degree of salinity likely to be encountered (the freezing point of sea water is above 25° Fahr.), and would generally make the freezing effective, that is, result in a continuous frozen wall in cases where the pipes diverged to some extent from the correct position, or where there might be a material flow of water through the earth. In fact, the writer believes that practically all the recorded difficulties experienced in the use of the process, excepting that from leaky pipes, could have been overcome by the circulation of cold liquid at such low temperatures as are entirely practicable with the proper refrigerating machinery now available.

The work mentioned at the Speedway, in New York City, was an experiment, made largely at the writer's expense, to test the feasibility of utilizing a compressor, belonging to an ordinary pneumatic plant, by adding a cooler and an expansion cylinder to the air compressor, thus converting it into an air ice machine and doing the freezing by a circulation of cold air. The ice machine thus made, and the cold-air circulation, proved inadequate.

With reference to the application of the freezing process to tunnels, the writer has devised a method of construction which does not necessitate the use of pipes inserted in the ground, but, in its simplest form, is the enlargement of a small preliminary or pilot tunnel by maintaining a very low temperature in it, as is done in refrigeration establishments, freezing outwardly from it, and enlarging it by excavation of the inner portion of the frozen material. The pilot tunnel, say of 7 ft. diameter, may be built by some other process for a cost

usually less than one-tenth of the cost of a full-sized railroad tunnel; or, this pilot tunnel itself may be built by the freezing process by keeping its apex in a conical shape, using a pilot pipe at the apex and excessively low temperatures in this apex, so as to freeze rapidly. Mr. Sooysmith.

As the writer introduced the freezing process into the United States, and has been most identified with its use and advocacy, his explanation of why the process has not been used more extensively here may be of interest: As utilized, up to the present time, the method of freezing has required the insertion, into the mass to be frozen, of pipes put down long distances and which have to be tight and so nearly in the position planned as to call for much skill and outlay for the attainment of these results. Then, for small works, the cost and difficulty of providing an ice machine has often been very considerable. In Europe, the mineral deposits that were easy to reach had been already exploited, whereas, in this country, there have been so many deposits still easily attainable that there has been, heretofore, little call for shafts where the difficulties would be very costly to overcome. Thus the business offering has not been sufficient to develop a contracting organization fully equipped for doing work in this way.

Recently, the demand for tunnels under the rivers surrounding Manhattan Island has made a field of large undertakings through water-bearing material, where the magnitude of each enterprise would be so great that the cost of the best engineering skill, and refrigerating plant adequate for the maintenance of the lowest practicable temperatures, could be incurred economically. The barrier to the use of the process for the construction of tunnels has been the difficulty of getting pipes into the necessary position under the river to do the freezing, all of which difficulty, the writer believes, is avoided by the use of the pilot tunnel, as mentioned. In the case of a tunnel under a river, there are numbers of factors to be considered, as, the proportion of water in the soil, conductivity, specific heat, movement of ground-water, proximity of river bed and flowing water, and other considerations of less importance. These matters have been fully considered, and can be provided for. The writer will not enter into them at present, as it would require more extensive treatment than could be given in this discussion. This tunneling procedure, as well as an improved method of constructing shafts, will form the subject of a paper which the writer intends to submit at a later date.

JAMES H. BRACE, ASSOC. M. AM. SOC. C. E. (by letter).—In preparing this paper it was the writer's purpose to show where information was lacking, as well as to summarize that available. Many of the uncertainties in regard to the use of the freezing process at various points could be cleared up by those who were in direct charge of the work. The writer is somewhat disappointed that the discussion has not brought out more particulars in this respect. Mr. Brace.

Mr. Brace. There are many factors yet to be determined before accurate estimates can be made in advance as to the time required for, and the cost of, any particular work. Theoretical study will prove very valuable in pointing out the nature of the problem and the elements for which the values are to be determined. The values of these elements, however, can only be determined by observations in actual practice, supplemented by laboratory experiments. Those in charge of the work at the Vicq Shafts made a start in the right direction, and some information was also collected at the Chapin Mine.

In planning future operations by the freezing method, it is to be hoped that a detailed system of observations for determining the unknown factors will be adopted in advance and then carried out faithfully. In this way the freezing process can soon be placed on a definite basis, and will command the confidence it undoubtedly deserves in certain classes of work.

Both Mr. Sooysmith and Mr. Campbell suggest the use of lower temperatures, and Mr. Sooysmith mentions one instance where calcium chloride brine was circulated at — 30° Fahr. It would be interesting to determine the lowest temperature at which the brine can be circulated, as this will probably determine the lowest limit at which this method can be used, rather than the capacity of the ice machine.

It may be well to note that, within the last few weeks, the freezing process has been used, on a small scale at St. Louis, to stop a leak in a steel coffer-dam. The published accounts are rather meager, but, apparently, the attempt was successful.

In conclusion, the writer will look forward with much interest for the appearance of the paper promised by Mr. Sooysmith, and trusts that it will not be long delayed.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 976.

THE SUBSTRUCTURE OF MARSH RIVER BRIDGE.*

By Herbert J. Wild, Jun. Am. Soc. C. E.

With Discussion by George B. Francis, M. Am. Soc. C. E.

The coast of Maine, from the mouth of the Kennebec River to the mouth of the Penobscot, presents a series of bays, inlets and estuaries. Indeed, the United States Geological Survey has selected the quadrangle in which Marsh River appears (the Boothbay Sheet) as an example of typical fiord coast line.

When the Knox and Lincoln Railway was built, in the early Seventies, its general direction was parallel to and about 12 miles distant from the coast. This location made it necessary to cross tidal waters five times in the 47 miles from the Kennebec River to Rockland. All these openings were spanned originally by wooden bridges, which have since been fitted with steel swing draws. This paper is a description of the substructure of the fourth of these to be replaced by a modern steel structure.

Marsh River Bridge, in the Town of Newcastle, spans a tidal branch of the Sheepscot River, and derives its name from its banks, which are the typical mud flats bordered with marsh grass.

The crossing is practically 600 ft. in width, but embankments have reduced the span of the present structure to 554.5 ft.

* Presented at the meeting of March 2d, 1904.

In August, 1902, the writer made surveys of the site of the old bridge, under the direction of Benjamin W. Guppy, M. Am. Soc. C. E., Bridge Engineer of the Maine Central Railroad. The designs of the new bridge, both substructure and superstructure, were developed by Mr. Guppy and approved by Mr. T. L. Dunn, Chief Engineer, the following winter.

On March 14th, 1902, application was made to the Secretary of War for permission:

"To substitute a steel plate-girder bridge on stone piers for the wooden trestle bridge on wood cribs and piles, built 1892 to 1896, which replaced the wooden bridge originally constructed about 1872."

The contract for the substructure was awarded to Ellis and Buswell, of Woburn, Mass., and the writer was assigned as Resident Engineer in Charge.

Work was begun on May 19th, and completed on December 10th, 1903.

The design called for six piers and two abutments of first-class ashlar granite masonry, with foundations of piles and concrete, as shown on Fig. 1, the general drawing, which also shows the old wooden bridge.

As will be seen by this drawing, the plan was to construct the new masonry without interfering with the passage of trains. The specifications also called for the construction of Pier No. 4 last, in order that the channel might be blocked and navigation impeded for the shortest time possible.

Abutments.—Work on the west abutment was begun first. One trestle bent of the old bridge came directly on the abutment site, as shown on the general drawing. Its removal was effected by driving a bent of six piles just east of the front line of the foundation, and a bent of six piles west of the rear line of the foundation. The track was then supported by three 8 x 14-in. track sticks under each rail, with a clear span of 21 ft. The regular stringers were blocked up on the new bents, and the interfering ones removed.

The excavation was carried to Elevation 43, or 1 ft. above low water. (All elevations are from an arbitrary datum.) The original plan was to drive piles here, as in all the other foundations. Hydraulic

GENERAL PLAN OF
MARSH RIVER BRIDGE,
ON KNOX AND LINCOLN BRANCH OF MAINE CENTRAL RAILROAD,
3/10 MILE EAST OF SOUTH NEWCASTLE, ME.

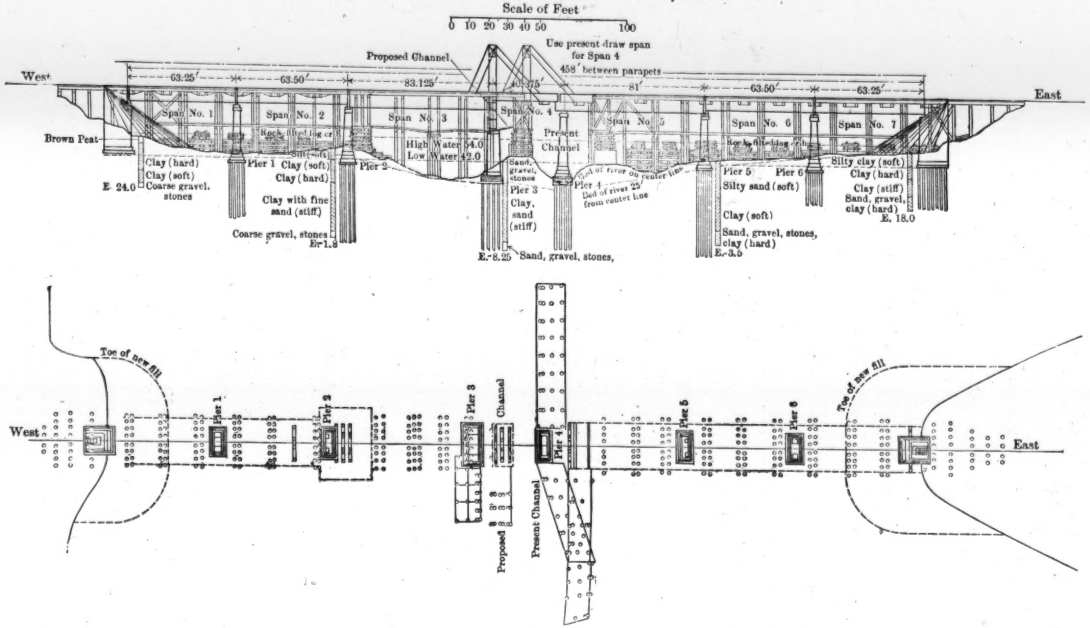


FIG. 1.

borings had been made in various localities, and the results are shown on the general drawing. When the excavation reached Elevation 43, a very firm, hard clay was encountered. So promising did it seem for a good foundation in itself that it was tested for bearing power. A 12 x 12-in. timber was arranged with a free vertical motion and loaded with a weight of 9 860 lbs., with no settlement. As this was largely in excess of the unit load the finished foundation was to bear, it was deemed safe to build without the piles, which was done. The east abutment was a duplicate of the western one, with the exception that the foundation piles were driven as shown.

One bent of the old bridge was directly on the east abutment site, and the first bent east was in danger of being undermined by the excavation. A bent of six piles was driven in front of the abutment site and a bent of four piles east of the threatened bent. The track was carried on three 8 x 14-in. sticks under each rail. The two trestle bents were removed and a third bent of four piles was driven to replace the easterly or threatened bent, narrowing the clear span to 20.3 ft.

On the west abutment, the excavated material deposited in front was sufficient to keep out the water. Lower ground adjoining the east abutment made some sort of a coffer-dam necessary. This was built with the clay excavated, faced and capped with turfs of marsh grass. After the collapse of a small section, at the start, a dam of sufficient height and strength was easily and quickly built. Beyond being overflowed by an extra high run of tides in August, this dam served its purpose admirably, with a minimum amount of leakage. A ditch pump handled all the water in the west abutment. A 6-in. centrifugal pump was used at the east abutment to clear the pit when necessary, but a smaller pump would have answered as well.

The abutments were laid with a boom derrick erected on the fill near the ends of the bridge. The general character of the masonry, and the method of carrying the track are shown in Fig. 1, Plate XIX.

Piers.—Six piers were contemplated, in four of which, *i. e.*, Piers 1, 2, 5 and 6, the piles were cut off at Elevation 40, or 2 ft. below low water. The construction of these piers will be described first. The piles in Piers 3 and 4 were cut off at Elevation 30. Their construction will be taken up separately.

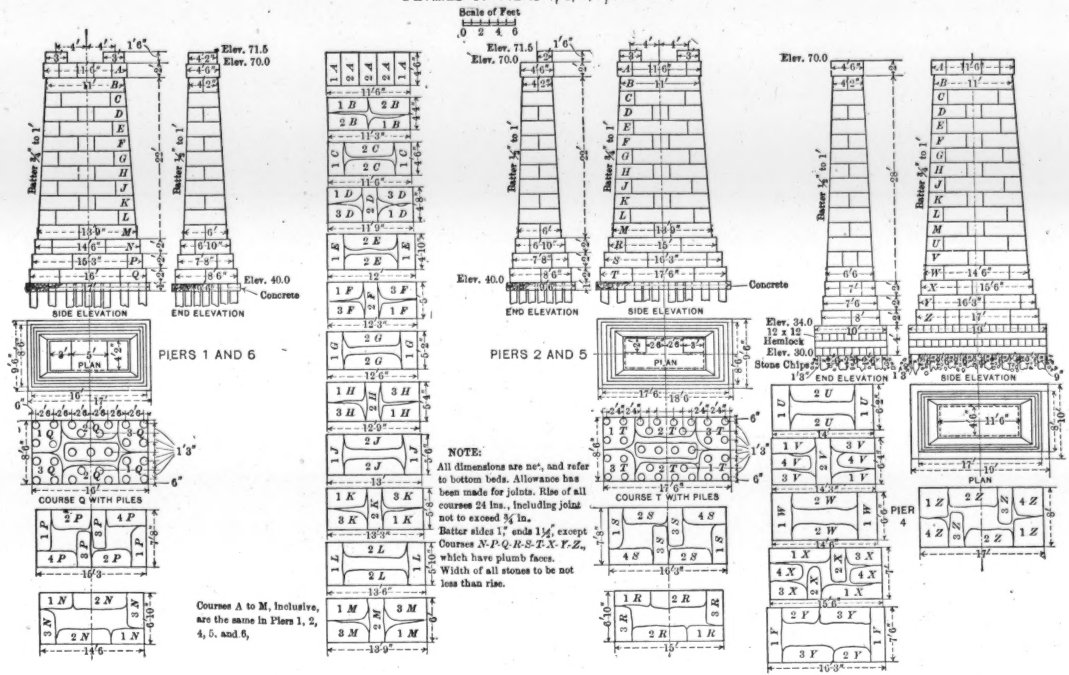

MARSH RIVER BRIDGE
DETAILS OF PIERS 1, 2, 4, 5, AND 6
Scale of Feet
0 2 4 6
PIERS 1 AND 6
PIERS 2 AND 5
PIER 4
SIDE ELEVATION
END ELEVATION
PLAN
COURSE Q WITH PILES
COURSE T WITH PILES
Batter 1/4" to 1'
Elev. 71.5
Elev. 70.0
Elev. 40.0
Concrete
Elev. 34.0
12 x 12 Hemlock
Elev. 30.0
Stone Chips
NOTE:
All dimensions are net, and refer to bottom beds. Allowance has been made for joints. Rise of all courses 24 ins., including joint not to exceed 3/4 in.
Batter sides 1" ends 1 1/2", except Courses N-P-Q-R-S-T-X-Y-Z, which have plumb faces.
Width of all stones to be not less than rise.
Courses A to M, Inclusive, are the same in Piers 1, 2, 4, 5, and 6,

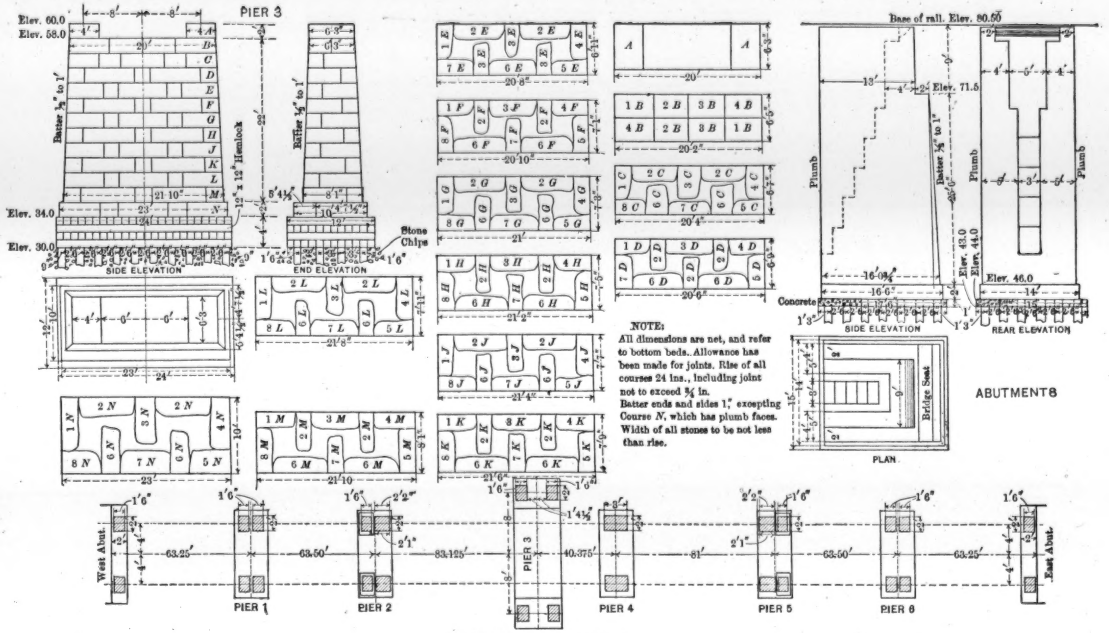
FIG. 2.

For Piers 1, 5 and 6, no track preparation was necessary. For Pier 2, the first bents, both to the east and west, were removed, and the crib shown on the east side was cut down close to the middle bent of the group of three bents shown on the plan. A bent of four piles was driven west of the group of two, and the track was carried on two 12 x 12-in. track sticks under each rail, with a span of 15.3 ft.

The sites of Piers 1, 2, 5 and 6 were directly over the old cribwork of the original bridge. The material to be excavated consisted of old cribwork, filled with rocks and mud from the flats. The nature of this material is indicated in Fig. 2, Plate XIX, a view of the old bridge looking east from the channel. This excavation was the hardest and most tedious part of these four piers. It was only possible to work on half tide or low tide. The old cribwork was so sound that it was necessary to cut nearly every timber, it being impossible to start them with the car derrick. Piers 1, 2 and 6 were excavated and the pits cribbed with 2-in. plank. Pier 5 was in deeper water on the north side, and it was necessary to drive a coffer-dam of 3-in. grooved and splined timber. Small boulders under the dam were removed by grappling for them by means of iron rods with hooks on the ends. After loosening them they were brought to the surface by workmen diving for them, the water being about 5½ ft. deep. This was a crude method, but, owing to the small quantity of rock to be moved, it was quite effective.

The excavated logs and rocks were used to build temporary cribwork, normal to the bridge, to carry the rear of the pile-driver. The driver was supported beneath the bridge on a platform of 12 x 12-in. timbers built on the adjacent bents of the old bridge. The driver was mounted on 10-in. hollow iron rolls running under a grooved carriage. Motion sideways was secured by sliding the carriage on the rolls by a rope rove from the carriage to the old bridge and to the drum of the engine.

The longest pile that could be put in the driver was about 32.7 ft. When longer ones were necessary, they were driven from the track by the derrick car. For this purpose, leads, 26 ft. long, without a top or bottom, were made. The piles were placed in position through openings in the ties. The leads were then raised by the pile-runner and placed on the pile, the whole being held in place by watch-tackles. The hammer was then put into the top of the leads and manipulated by the regular fall of the car's hoisting engine.



LOCATION OF BED PLATES

MARSH RIVER BRIDGE
DETAILS OF PIER 3, ABUTMENTS, AND PLATES

FIG. 3.

When the piles had been driven to such an elevation that the driver mounted below could be rolled over them, the hanging leads were removed and the piles followed home from below. A view of this is shown in Fig. 1, Plate XX, which also shows Pier 1, and the old cribs, etc., at low water.

After the piles were driven, the pits were pumped out by a 6-in. centrifugal pump, the piles were cut off and the concrete laid. The masonry was laid, with a self-propelling car derrick, from the track. All stones for the piers came to the work cut and ready to lower into place, as shown by the detailed plans. On these piers the cribs and coffer-dams were carried only to about Elevation 44; there being no attempt to work on the masonry except at low water.

Piers 3 and 4.—To carry the track during the construction of Pier 3, a bent of six piles was driven west of the proposed pier, and a trestle bent erected on a projection of the west side of the old draw pier. The bents which are shown on the general drawing as interfering with the construction were removed, the track being carried on three 8 x 14-in. sticks under each rail, in addition to the regular beams, the clear span being 17.1 ft.

The excavation for Piers 3 and 4 was largely old rip-rap with some flats mud. It was removed by an orange-peel dredge bucket hung to a boom attached to the adjacent bents of the old bridge. The material was discharged on a scow which was run beneath the bucket. It was only possible to work from low to half tide, lack of head room not permitting the bucket to be raised high to deposit on the scow at high water. The engine was mounted on a platform on the old cribbing.

In excavating for Pier 3, considerable old cribwork was encountered. For its removal, the power chisel, shown in detail in Fig. 4, was constructed. This chisel was bolted to a pile butt, and after being placed over a timber to be removed, was driven through it in a manner similar to that described for driving long piles. Some of the timbers thus detached were too water-soaked to come to the surface. They were raised by a pair of large timber hooks, made of § x 3-in. iron, secured to the ends of handles, 15 ft. long, and manipulated from the scow. After a successful hitch had been made with this set of mammoth pinchers, the timbers were lifted by the car derrick. Both the hooks and the chisel are shown in Fig. 2, Plate XX.

FIG. 1.—EAST ABUTMENT, MARSH RIVER BRIDGE.

FIG. 2.—THE OLD MARSH RIVER BRIDGE, LOOKING EAST.

An improvement on the chisel could have been made by giving the blade a greater length than 14 ins., and a little greater depth than 4½ ins., thus making it easier to place it on a sunken timber.

The piles in Pier 3 were driven from a platform supported by the bents of the old bridge. Here it was necessary to use two followers. A follower 10 ft. long was used first to drive the pile to a point where a 25-ft. follower could be used to drive the pile home; this being necessary because the hammer could be dropped only 19 ft. without leaving the bottom of the leads. The piles in Pier 4 were driven from a scow in the usual way.

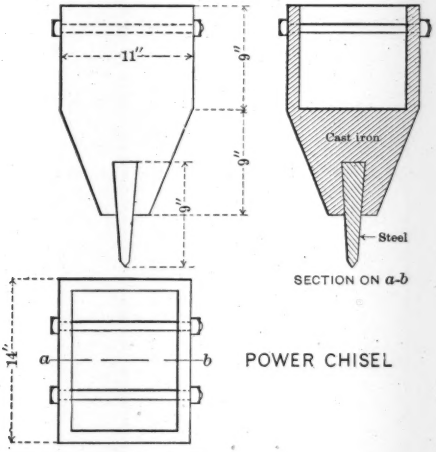


FIG. 4.

The piles shown on the general drawing are to scale, as to the depths driven, and thus show the approach of actual driving to the borings made by the hydraulic process.

Cutting the piles off at Elevation 30, 12 ft. below low water, gave more trouble and caused more expense and delay than any other part of the work, especially in Pier 3. In Pier 4, they were cut off with a 34-in. horizontal circular saw mounted on a 4-in. shaft, 27 ft. long. The shaft was attached to an upright 12 x 12-in. timber with three bearings 6 ft. apart. The stick was fastened in the leads of the pile-driver, mounted on a scow. The shaft was suspended by a tackle

with a swivel which admitted of its rotation and vertical adjustment through its bearings. The power was transmitted to the shaft by a 6-in. belt from the pile-driving engine. The saw was set in position by a submarine diver, and kept at the correct elevation by aligning the shaft with a wye-level. The saw was fed against the shaft by shifting the scow with bow and stern lines and watch-tackles. The extreme swiftness of the tide made it impracticable to work except from low to half tide. Much trouble was caused by getting the saw into two piles at once, which caused binding. This happened once during a rapidly rising tide, and resulted in a bent shaft. Trouble was caused, also, by continual slipping of the belt. On the whole, the efficiency of the outfit was very disappointing. The thirty-two piles were cut off in six days, including all delays for tide, etc., and two days lost by the bent shaft. The diver reported a very creditable result at the finish. For cutting off the piles in Pier 3, the same outfit was tried. The leads and engine, in this case, were mounted on rolls, as in the pile-driving. The bearing boxes were changed to 10 ft. apart, and a 10-in. belt was substituted for the 6-in. belt. It was also necessary to remove the side braces from the leads to permit work to be done close to the old bridge. When the engine was started, the vibration of the platform was so great that the leads and engine traveled sideways on the rolls, and even the rolls shifted. The result was that the saw, on being fed into a pile, had a swaying motion which jammed the teeth hard into the wood, and stopped the saw with a sharp jerk. After a series of attempts to remedy the trouble, the saw was abandoned. The principal difficulty in the use of the horizontal saw seemed to be due to four causes: First, in feeding the saw gradually into a pile which was entirely invisible; second, the extreme swiftness of the tide; third, the continued tendency of the belt to slip; fourth, lack of engine power (an 8 x 12-in. single-cylinder engine was used).

On abandoning the power saw in Pier 3, the piles were cut by two submarine divers, the cut-off being given them on each pile by a 28-ft. iron pipe, 1½ ins. in diameter, and a wye-level. The forty piles were cut off in seven days, on four days of which only one diver worked. The divers worked from three to four hours daily. Their principal trouble seemed to be in getting a proper saw. Several kinds were tried, but none seemed to be entirely suitable for the work.

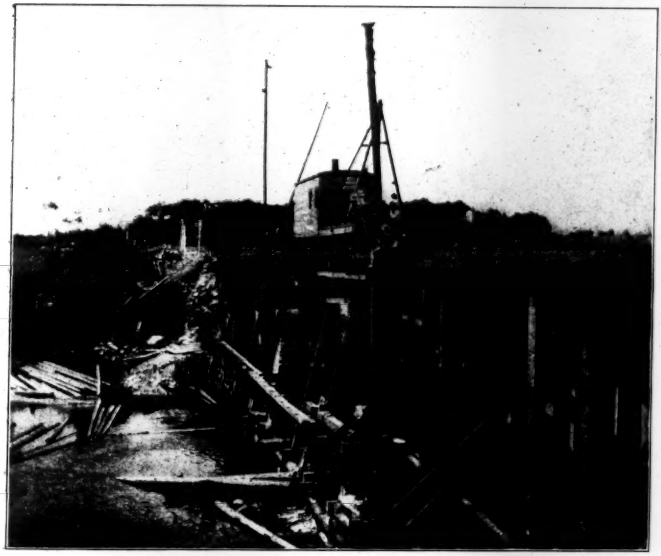


FIG. 1.—DRIVING LONG PILES, MARSH RIVER BRIDGE.

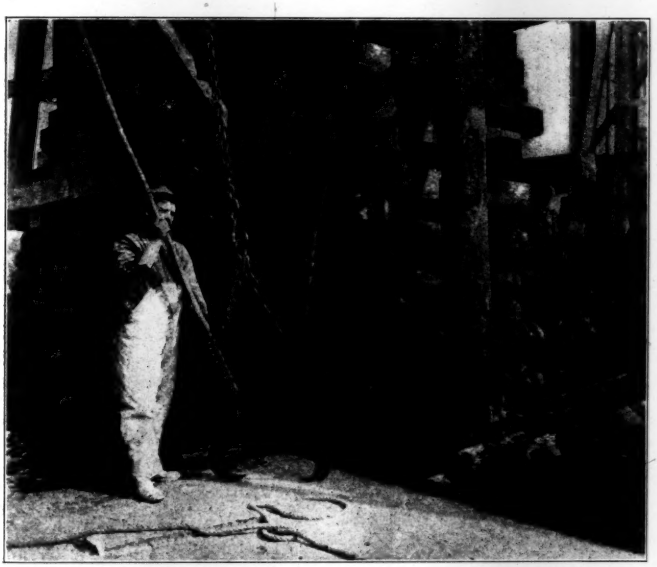


FIG. 2.—TIMBER GRAPPLES AND POWER CHISEL, USED AT MARSH RIVER BRIDGE.

After the piles were cut off, stone chips were dumped among the pile heads and leveled by the divers. These chips were rated as concrete.

The method first proposed for the construction of Piers 3 and 4 was by a floating caisson, the bottom of which was to be the grillage of 12-in. timbers shown on the detailed plans, the sides to be removable. The writer suggested as a substitute the device which was finally used. For Pier 4 a bent of four piles was driven on each side of the site of the pier and cut off at Elevation 45.17, as shown in Fig. 5. These piles were capped with 12 x 12-in. hard pine. Three stringers

DEVICE FOR CONSTRUCTING PIER 4. MARSH RIVER BRIDGE. SOUTH NEWCASTLE, ME.

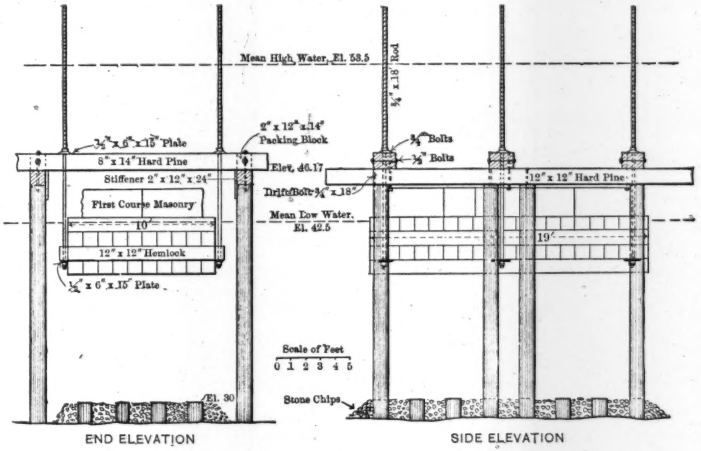


FIG. 5.

of two 8 x 14-in. hard pine, spaced 2 ins. apart, were set at such points as to be directly over the lowering rods when the grillage should be over its proper position on the piles. The two end and center sticks of the next to the bottom course of the 12-in. timbers in the grillage were given a projection of 6 ins. on each side; each timber being cut out 1 in., thus making a 2-in. slot 6 ins. deep. The lowering rods, six in number, were of 1¾-in. iron, 10 ft. at the top being threaded. The grillage was floated into position beneath the stringers on a falling tide, and the rods were inserted in the slots and plumbed. Two nuts

and a ½ x 6 x 15-in. plate washer were used on the bottom of each rod, and a washer of the same size, with the addition of a 9-in. round washer and one nut, was set on top of the stringers. After placing in position, a course of stone was laid and the upper nuts slacked as much as possible before the rising tide flooded the work. Wedges were placed between the top of the masonry and the stringers as a preventive of possible lateral motion. This operation was repeated at each tide, and on the seventh tide the grillage rested on the piles, and was practically correct as to both level and position. Darkness prevented utilizing the whole of each tide, or the pier would have been lowered in five tides, *i. e.*, one course to a tide.

After the grillage rested on the piles, the stringers, rods, etc., were removed and used again in Pier 3 (which, contrary to expectation, was built after and not before Pier 4). The remainder of Pier 4 was built up to high water as tide work.

Pier 3 was constructed by the same means as Pier 4. Here, however, one nut was left off the bottom of each rod, and the rods were headed up. Two polished steel washers were also inserted between the lowering nut and the 9-in. cast washer. An improvement on this device could be made by inserting a ball-bearing washer below the lowering nut. Also, the use of three connected ratchet wrenches on each side would have assured an equal turning of all nuts, and obviated an occasional binding. Pier 3 was landed on the piles in ten tides, and then completed rapidly.

The pier masonry and excavation, throughout, was carried on with a self-propelling derrick-car working under flag protection. This car was the property of the contractors, Ellis and Buswell, and on this work, especially, was a valuable piece of machinery, as it enabled them to bring out stock to any pier, and work directly over it. From the beginning of the work until the middle of October there were thirteen trains during working hours, and after that time, seven trains. Waiting for trains caused some delay, especially at low tide.

Table No. 1 is a force account, in connection with which a word of explanation is due. The small amount under "Laborers," in proportion to men of higher wages, is due to the fact that the masons, blacksmiths, etc., worked on excavations, pile-driving and other work, as time or tide made necessary, it being impossible to work a large crew to advantage.

TABLE No. 1.—FORCE ACCOUNT.

	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Totals.
Foreman	11	25	26	28	27	28	28	9	182
Stone cutters	.....	.....	.....	24	32	.....	.....	.....	56
Masons	30	90	109	101	78	56	69	27	560
Engine runners	11	41	38	81	64	52	28	9	324
Blacksmith	11	25	25	28	26	28	28	9	180
Derrick men	11	41	28	28	26	28	25	9	196
Mortar men	.....	17	25	28	26	9	15	9	129
Laborers	27	64	50	78	79	72	90	26	486
Divers	.....	.....	.....	.....	.....	3	20	4	27
Diver's tender	.....	.....	.....	.....	.....	3	20	4	27
Flag	.....	20	25	25	26	27	25	9	157
Totals	101	323	326	421	384	306	348	115	2 324

All cement used on the work was subjected to the accelerated tests recommended in the report of the Special Committee on Uniform Tests of Cement.* The water of the river, which was used on the work, was also used in the tests.

The final estimate of work done is given in Table No. 2.

TABLE No. 2.—FINAL ESTIMATE.

Excavation of earth and old cribs	1 397 cu. yds.
Piles, below cut-off	8 068 lin. ft.
Concrete and chips in Piers 3 and 4	84 cu. yds.
Timber in grillage in Piers 3 and 4	22 944 ft., B. M.
Pier masonry	579 cu. yds.
Abutment masonry	463 cu. yds.

The filling about the two new abutments was deposited by the railroad company, and consisted of limestone chips from the quarries at Rockland, 33 miles distant.

The writer regrets that he is unable to show any photographs of the method of lowering the masonry of Piers 3 and 4, as an accident in the darkroom caused all the negatives to be light struck.

The nuts were turned by key wrenches with handles about 4 ft. long. In both piers, five courses were in place before they rested finally on the pile heads. Most of the time, it required two men on each wrench. With the improvements mentioned, and where the current was less rapid, this method would be most successful; even

* *Proceedings*, Am. Soc. C. E., January, 1903.

in this work, with the current and heavy load, the lowering was done without a hitch.

A question as to the probability of sea worms attacking the timber grillage has been asked. The timber cribs and piles of the old bridge, on examination, do not show a single instance of any attack of this nature, and there seem to be no reason for anticipating any.

The swiftness of the tide has been mentioned. By an examination of the general plan it will be seen that the cribs of the old bridge contracted the river to a large extent.

There were no means at hand to determine the actual current velocity, but it was swift enough to make all water work exceedingly difficult.

DISCUSSION.

GEORGE B. FRANCIS, M. AM. SOC. C. E.—The Society is indebted to the author for his good description of the work done. The paper describes a method of lowering a masonry pier, built on a timber platform, to its final bearing on a pile foundation, by long screw-bolts, with the threaded end of the bolts at the top, the lowering having been effected by turning nuts on these bolts. In 1888 the center pier of a single-track draw-bridge at Oakland Beach, R. I., was placed, in this manner, under the speaker's direction, with the exception that the rods were reversed, *i. e.*, placed with the threaded ends down and with the nuts boxed into the underside of the timber platform. The pier was lowered by turning the heads of the bolts, instead of the nuts. This was necessary because the track grade was near the water and the old draw was frequently turned for the passage of boats; if the bolts had been set with the screw ends up it would have been impossible to swing the draw. Mr. Francis.

In 1892 the end piers of the double-track draw-bridge, on the Shore Line, at Mystic, Conn., were lowered in the manner described by the author. Under certain conditions this method proves very economical and effective, particularly where the amount of lowering required is moderate and the difficulties of coffer-damming are great.

The speaker has had no especial difficulty in sawing off piles under water with a circular saw on a shaft placed in pile-driver leaders on a scow. The binding of the saw experienced by the author was probably due to the light scow used. It will probably be remembered that A. P. Boller, M. Am. Soc. C. E., had piles cut off in this manner, about 50 ft. under water, at the Thames River Bridge, New London, Conn. It is difficult to saw the piles off at an even grade, owing to the rise and fall of the tide and to some surging of the scow. Even at the moderate depth of 6 ft., the writer found it difficult to saw off a cluster of piles within 1 in. of the proper grade line (measured up or down), or within a range of 2 ins.

At great depths, like 50 ft., records obtained with a steel tape line have shown that it is difficult to cut off piles within $2\frac{1}{2}$ ins. of the proper grade line, or within a range of 5 ins. In such cases it is expected that the weight of the masonry on the platform grillage will crush the timber until the platform comes to an even bearing on the piles.

Exception has been taken to the author's conclusion that it is preferable to use divers, rather than machinery, to cut off piles under water, and the opinion has been expressed that divers are about the most unreliable agents that can be used. The speaker thinks that much depends on the experience of the divers employed. No doubt, some are unskilled on construction work under water who would be

Mr. Francis. skilled on other work. In constructing the Boston Terminal, occasion required the removal of some crib timber buried in gravel filling under water, and certain local divers were employed for the work. They spent many weeks in the attempt, making very little progress, and then practically confessed their inability to do the work. In despair, the contractor for the terminal work turned to the writer for advice, and was directed to send for a certain diver experienced in wrecking and construction work under water. This was done, and, although the cost was large, the work was executed promptly. Such a diver will succeed in this class of work where many others fail.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 977.

THE LOCATION OF THE KNOXVILLE, LA FOLLETTE AND JELLICO RAILROAD, OF THE LOUISVILLE AND NASHVILLE SYSTEM.*

By W. D. Taylor, M. Am. Soc. C. E.

With Discussion by Messrs. Emile Low, William P. Watson, E. J. Beard, Walter Watson, William G. Raymond, F. Lavis, W. H. Coverdale, Harvey Linton, W. D. Forsythe and W. D. Taylor.

There are experienced field engineers to whom the perusal of this paper will be a loss of time. Many engineering papers deal with the special characteristics of special structures, to the exclusion of broader questions. The experienced engineer spends most of his time in thinking out the details of such structures, and, when he writes an account of his work, assumes that his readers will grasp at once the broader questions involved, and will be interested only in the special designs which have taxed his own ingenuity most. This would be all very well if his writing fell only under the eyes of experienced readers; but, probably, ten inexperienced readers, ten students, perhaps in search of information, will read his article carefully where one busy and experienced engineer gives it hasty perusal. For the inexperienced reader, the broad questions affecting the whole

* Presented at the meeting of March 16th, 1904.

proposition are oftentimes of more value than a minute description of the details of special structures. For him a discussion of the economic questions which determined that the whole project of construction was advisable may be of more importance than the most intricate and learned calculation upon the strength or efficiency of special structural parts; or, to be still more specific, the reasons for building a road in one valley rather than in another offering a shorter route may be more instructive to the majority of readers than any or all of the detailed masonry plans worked out in the construction of the road. In this paper the writer has attempted to set forth the considerations which determined each important step in the lay-out of what he believes is the most important engineering work undertaken in recent years in the section of country in which this road lies. The paper does not even mention the part of the work on which the writer spent most of his force; but, in his judgment, it includes an account of all the best work accomplished on this undertaking, both by himself and by those working over and under him.

In March, 1902, the writer was tendered the position of engineer of construction of the Knoxville, La Follette and Jellico Railroad, with the assurance that the line was to be constructed "through difficult country."

The Louisville and Nashville Railroad Company has a line from Cincinnati to Jellico and from Louisville to Jellico (see Fig. 1). At this time, the company had just acquired control of the Atlanta, Knoxville and Northern Railroad. For some years previous, it had operated the Georgia Railroad under a joint lease with another company. For a long while, it had had a large, if not a controlling, interest in the Atlanta and West Point Railroad and in the Western Railway of Alabama. Thus the system was in entire or partial control of some 900 miles of road to the south of Knoxville, with which it had direct connection only at Montgomery, and connection through a controlled road at Atlanta. From Fig. 1 it will be seen that, in order to have these properties closely united to the body of the system, as well as to operate through trains from Cincinnati and Louisville through Knoxville to Atlanta, the construction of the gap from Jellico to Knoxville was necessary.

The order for the survey and construction of the line was not accompanied by any of the usual tentative conditions. A fair traffic

Terre Haute
INDIANA
To Chicago
OHIO
Cincinnati
Covington
Newport
Ohio R.
Maysville
Johnson
MONON
Louisville
To St. Louis
Evansville
Henderson
Owensboro
Paris
Bloomfield
Winchester
KENTUCKY
Rowland
Livingston
Greensburg
Penrod
Russellville
Glasgow
Corbin
Norton
Memphis Jc.
Princeton Jc.
Jellico
Middlesborough
To Memphis
Lebanon
Nashville
Bon Air
Clinton
TENNESSEE
Knoxville
Perryville
Columbia
Pikeville
Allens Ck.
Tullahoma
Tracy Cy.
NORTH CAROLINA
Flakney
Iron City
Chattanooga
Murphy
Sheffield
Stevenson
Blue Ridge
SOUTH CAROLINA
Tuscumbia
Decatur
Guntersville
Rome
Kingston
Gainesville
Attalla
Gadsden
Marietta
Atlanta
Social Circle
Union Pt.
Barnet
Camak
Birmingham
Anniston
GEORGIA
Tacoa
Talladega
Sylacauga
Blocton
Calera
Macon
Wetumpka
GEORGIA
Selma
WESTERN RY. OF ALABAMA
Montgomery
Camden
Georgiana
Florala
Geneva
Graceville
River Junction
Mobile
Pensacola
FLORIDA
GULF OF MEXICO
Louisville & Nashville Railroad
Lines Controlled by the L. & N.
Lines under construction.

FIG. 1.

could be safely counted on for the new line, from the day of its completion, without considering the large local traffic which could probably be developed along the route. Thus the chief executive of the road was warranted in authorizing expensive construction, if need be, to secure good grades and alignment.

The distance from Saxton, Kentucky, to Knoxville, Tennessee (see Fig. 2), on a straight line, is approximately 51 miles; *via* the Louisville and Nashville to Jellico, and the Knoxville and Ohio Branch of the Southern from Jellico to Knoxville, it is 68.3 miles. This branch of the Southern was built years ago, when, in the location of railways, little attention was paid to future economy in operation. The line, in places, has grades of 93 ft. to the mile, uncompensated over stiff curves, even near the tops of long ascents. Helper engines are used freely on the line, even on passenger trains. But the traffic in south-bound bituminous coal, from Jellico, Pioneer, La Follette, Coal Creek, Briceville and Oliver Springs, is large enough, it is said, to make this one of the best paying lines of that system.

The entire country between Saxton and Knoxville has been covered by the United States Geological Survey, and the maps of this Survey, though found to be very inaccurate as regards topography in country inaccessible by horse or wagon, were of untold value in choosing the routes for the line.

Waldens Ridge and Cumberland Mountain (See Fig. 2) divide the district into two parts, unlike in geography, topography, and geology. That portion of the country to the northwest of this ridge and mountain is part of the Cumberland plateau of the Appalachian province. The drainage, for the most part, is to the north and northwest into the Cumberland River. The portion of the district to the southeast of this divide is a part of the great valley of East Tennessee, and the general drainage is to the southwest into the Clinch and Tennessee Rivers. To the northwest of this dividing line, the surface rocks are Carboniferous; to the southeast they are Cambrian and Silurian. The country to the northwest of this line is so broken and rugged that it may be called the mountain district. Here the divides rise to elevations of from 2 500 to 3 600 ft., large areas being above 3 000 ft. The streams fall rapidly, from their sources, and emerge into the valleys at elevations of from 800 to 1 100 ft. These valleys are deep and narrow, and the slopes rise brokenly. The crests of the mountains

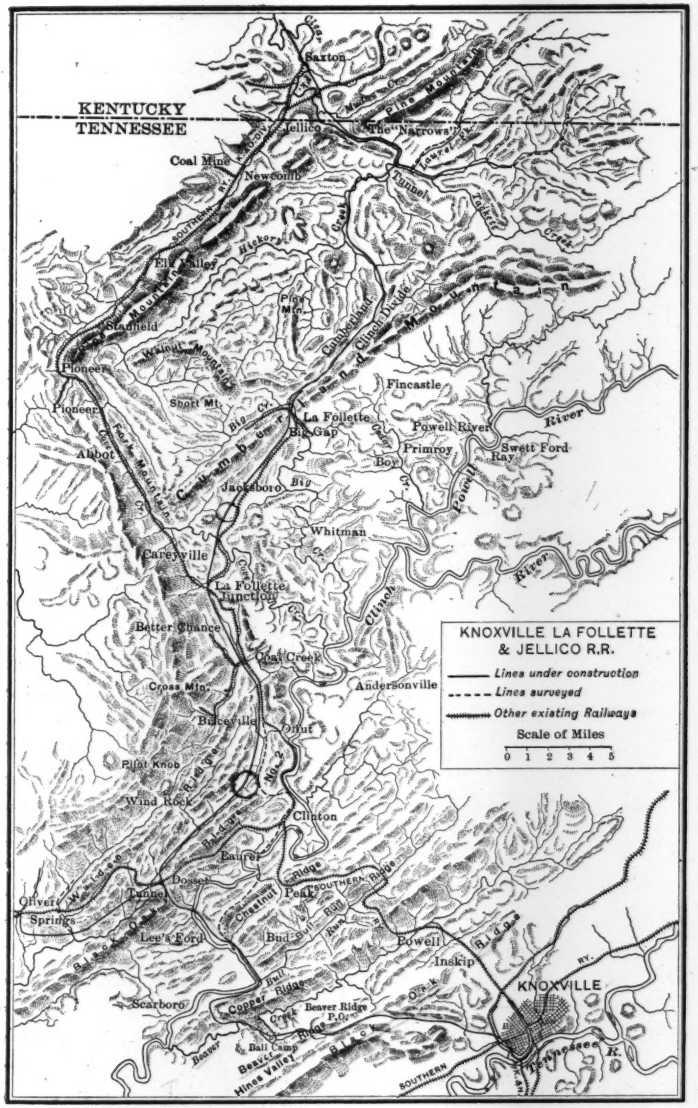

KENTUCKY
TENNESSEE
Saxton
Jellico
The "Narrows"
Coal Mine
Newcomb
Tunnel
Elk Valley
Stanfield
Pioneer
Pioneer
Albot
Fincastle
La Follette
Big Gap
Powell River
Primroy
Swett Ford
Ray
Boy
Jacksboro
Whitman
Careyville
La Follette Junction
Better Chance
Coal Creek
Andersonville
Cross Mtn.
Briceville
Pilot Knob
Wind Rock
Clinton
Lauret
Dossett
Tunnel
Oliver Springs
Lee's Ford
Bud
Powell
Inskip
KNOXVILLE
Scarboro
Copper Ridge
Beaver Ridge
Ball Camp
Hines Valley
Tennessee R.
KNOXVILLE LA FOLLETTE & JELLICO R.R.
Lines under construction
Lines surveyed
Other existing Railways
Scale of Miles
0 1 2 3 4 5

FIG. 2.

are narrow and flat, many of them being susceptible of cultivation; but, so far, the steep and rugged slopes have proved to be effectual barriers to the settlement of the district, except by the scattered cabins of mountaineers. There are homes in these mountains which have been occupied for generations, with no wheeled vehicle ever in use until roads were opened by the contractors for the construction of this railroad. The mountain ranges are cut in two by occasional streams, and thus all possible routes on reasonable gradients are well defined.

In the valley district, erosion has produced a series of long ridges separated by long, parallel and narrow valleys which follow closely the belts of rock. Their general direction is northeast and southwest, thus crossing the direction of the line of survey, which lay somewhat northwest and southeast. The surfaces of these small valleys are at elevations of from 800 to 1 100 ft., and the parallel ridges rise from 100 to 500 ft. above them. Some few of these ridges are cut in two by streams, but most of them are continuous for many miles. Copper Ridge, on the south side of the Clinch, which ridge is responsible for the second large détour to the westward in the line of road between Coal Creek and Knoxville, was such a continuous ridge. From the point where the road cuts through it, for 30 miles to the northeast, there is not a gap in which the crest is not more than 350 ft. above Bull Run Valley on the north side of it. The conditions as to continuity were somewhat similar in the case of the two Black Oak Ridges. Thus the problem of getting the best location was a more difficult one in the less rugged country.

Clear Fork River cuts through Pine Mountain in a gorge called "The Narrows," so rugged that no domestic animal had ever traversed it. It lay across the straight line joining Saxton and Knoxville. The stream, Big Creek, cuts through Cumberland Mountain at Big Creek Gap, which lies some 24 miles from Saxton and about 4 miles west of the direct route. Further, a branch line of road had been constructed through this gap (see Fig. 2), and to a connection with the Southern Railway near Careyville. At the gap, the mining town of La Follette, with limestone and sandstone quarries, coal and iron mines, coke ovens, a furnace, etc., had sprung up and in three years had grown from a village of less than 500 to a population of more than 6 000. Thus these two water gaps were objective points. In fact, the route through these points had often been explored, and at least two

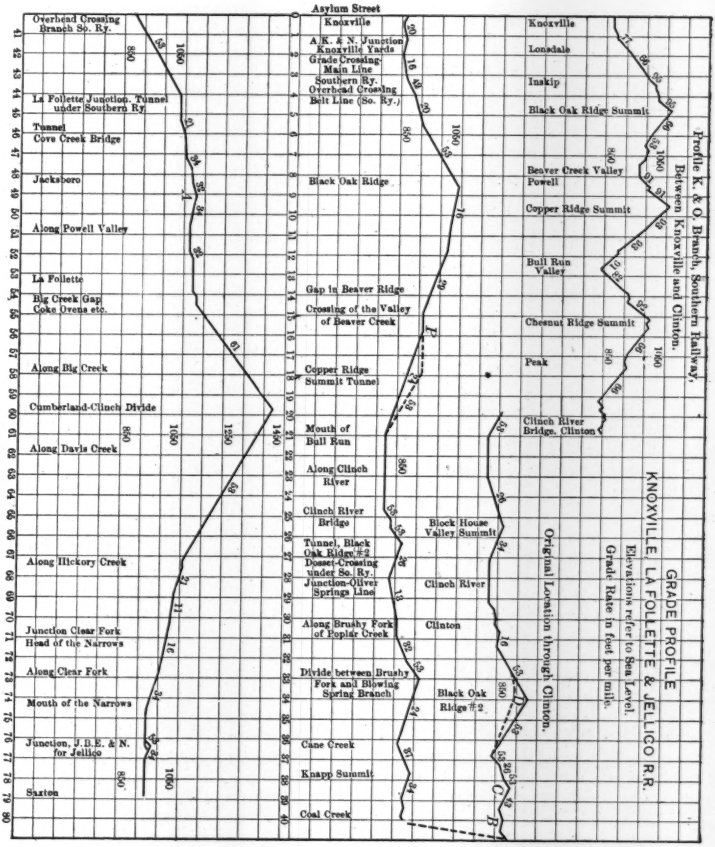

Profile K. & O. Branch, Southern Railway,
Between Knoxville and Clinton.
KNOXVILLE, LA FOLLETTE & JELLICO R.R.
GRADE PROFILE
Elevations refer to Sea Level.
Grade Rate in feet per mile.
Original Location through Clinton.
Asylum Street
Knoxville
A.K. & N. Junction Knoxville Yards
Grade Crossing Main Line Southern Ry.
Overhead Crossing Belt Line (So. Ry.)
Black Oak Ridge
Gap in Beaver Ridge
Crossing of the Valley of Beaver Creek
Copper Ridge Summit Tunnel
Mouth of Bull Run
Along Clinch River
Clinch River Bridge
Tunnel, Black Oak Ridge #2
Dosset-Crossing under So. Ry.
Junction-Oliver Springs Line
Along Brushy Fork of Poplar Creek
Divide between Brushy Fork and Blowing Spring Branch
Cane Creek
Knapp Summit
Coal Creek
Block House Valley Summit
Clinch River
Clinton
Black Oak Ridge #2
Knoxville
Lonsdale
Inskip
Black Oak Ridge Summit
Beaver Creek Valley
Powell
Copper Ridge Summit
Bull Run Valley
Chestnut Ridge Summit
Peak
Clinch River Bridge, Clinton
Overhead Crossing Branch So. Ry.
La Follette Junction. Tunnel under Southern Ry.
Tunnel
Cove Creek Bridge
Jacksboro
Along Powell Valley
La Follette
Big Creek Gap Coke Ovens etc.
Along Big Creek
Cumberland-Clinch Divide
Along Davis Creek
Along Hickory Creek
Junction Clear Fork Head of the Narrows
Along Clear Fork
Mouth of the Narrows
Junction, J.H.E. & N. for Jellico
Saxton
A
B
C

FIG. 3.

careful surveys had been made on it during the twenty years or more that the Louisville and Nashville Railroad had had under contemplation the extension of their Knoxville Division to the city for which the division was named. As there were watercourses leading from each of these points toward the other, the choice of the route joining them lay simply in deciding which slope of the narrow valleys offered the best support for the adopted gradient. But these valleys were so tortuous and narrow that in a distance of 3 miles on one of them it was necessary, even when using 10° curves with 300-ft. minimum tangents, to bridge the watercourse ten times and use three short tunnels.

It was plain, from the conformation of the country, that there would be long ascents and descents on whatever ruling grade was adopted. The chief local product of the country being bituminous coal, in great quantity and of good quality, the market for which could only be southward, since the country to the north and west was tributary to the Pittsburg coal fields and that to the east to the Virginia coal fields, it was good economy to spend more money to secure light grades against south-bound trains than for trains in the opposite direction.

The writer decided on 53-ft. compensated grades as the maximum, but an economical construction required the introduction of a 61-ft. grade on the long descent from the Cumberland-Clinch Divide to La Follette. The road was located from Saxton to this divide on this ruling grade, with an inconsiderable amount of adverse grade 2 miles from Saxton, at a point where its introduction was comparatively harmless.

An adverse grade has been defined as a grade pitching in the opposite direction from the general slope of the country. Of course, it means the introduction of just so much rise and fall which could be avoided.

The line from La Follette to Knoxville had not been reconnoitered. It seemed possible to get a line from the Cumberland-Clinch Divide down to the Clinch without adverse grade; but from the Clinch to Knoxville, across the short ridges and valleys, it seemed that the road would have to "rise and fall with the country," and the writer expected at the outset that the condensed profile of this part of the line would look like the teeth of a saw. From La Follette toward Knoxville, the narrow and tortuous valley of Big Creek seemed to offer the

most available route in the desired direction; but to get from the Clinch, at the mouth of Big Creek, across to Knoxville on 1% grades was out of the question, unless one made up his mind to make long tunnels through nearly all the ridges and to make high crossings over the narrow valleys. The difficulties and expense were so great that this route was never comtemplated seriously.

By going down Big Creek from La Follette to the Clinch, however, and then turning down the Clinch as far as the mouth of Bull Run, below Clinton, and by making three crossings of the Clinch, a route was possible, leading through Clinton, a town of 1 200 people, which gave no adverse grade from La Follette to Clinton and an adverse grade of only 48 ft., vertical, between Clinton and Bull Run, at a point about 4 miles southwest of Clinton where it was necessary to cut across country, leaving the river, to save distance and excessive curvature.

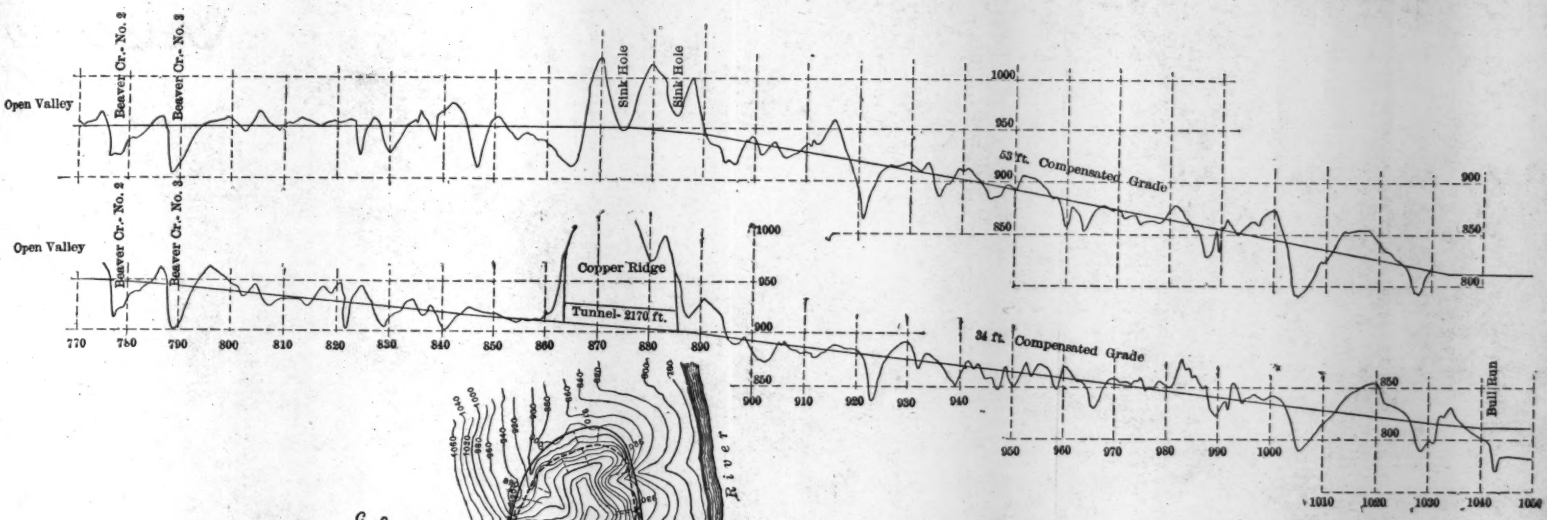
The absence of adverse grade was considered so advantageous on this route that a careful survey was made over all the difficult portions of it, and careful estimates were made of the value of its advantage and cost as compared with the route adopted finally; but the absence of adverse grade was its sole recommendation. There were many serious objections to it. First, to follow Big Creek Valley below La Follette required bad alignment, several short tunnels and expensive construction; second, the three crossings of the river involved expensive bridging; third, the distance from La Follette to Clinton was greater than by the route adopted finally; and, fourth, it led out and away from the coal fields, and through a country from which the timber had already been stripped and from which little development could be expected.

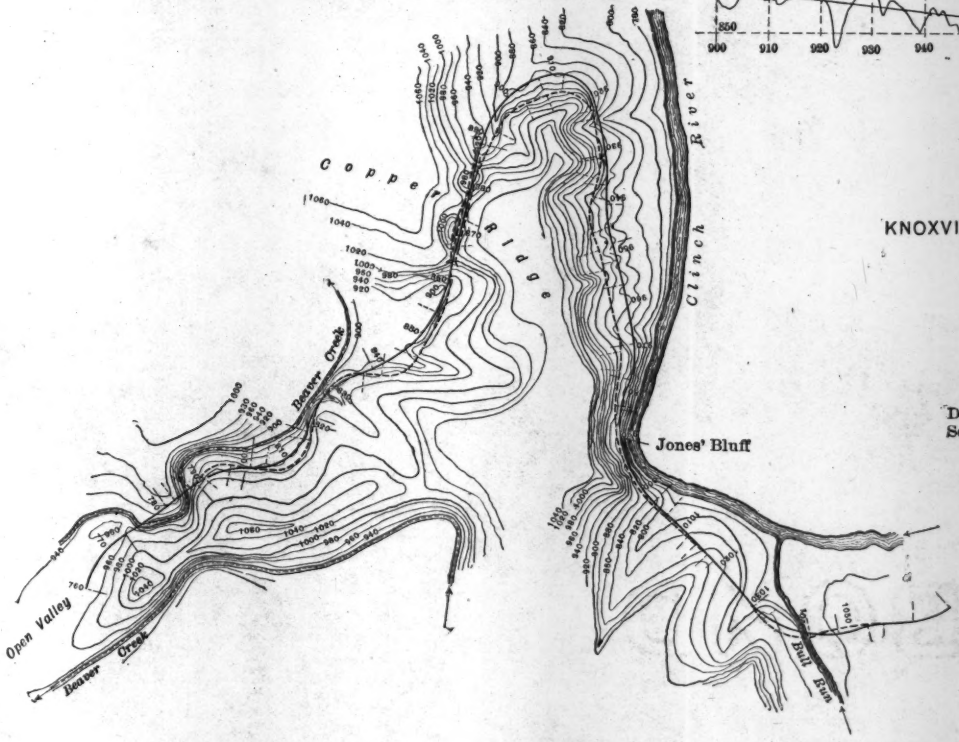
From La Follette to Clinton the route adopted originally followed very closely the line described previously as dividing the country into dissimilar sections. The route could only be adopted at the sacrifice of introducing a considerable amount of adverse grade at four different places (see Fig. 3 at *A*, *B*, *C*, and *D*). There were 28 ft., vertical, at *A*, 12 at *B*, 40 at *C*, and 132 at *D*. But the line was much cheaper than the Big Creek-Clinton line, afforded much better alignment, and, besides following the edge of the coal fields, passed through the town of Coal Creek, the center of a population of some 5 000 people, where coal mining had been carried on successfully for a quarter of a century.

The location from the mouth of Bull Run to Knoxville introduced more difficult problems than any other part of the line; and the writer believes that the combination of conditions which enabled this line from the Clinch to Knoxville to be laid out, on the original 53-ft. grades, without one foot of adverse grade and without a tunnel or high valley crossing, is unique and unusual in such rough country. The profile of the Southern Railway line from Clinton to Knoxville (see Fig. 3) is a fair representation of the necessary line that goes plunging across these ridges. An old locating engineer, of the Cincinnati Southern, years before, made a location survey from Harriman to Knoxville which passed through the same gap in Copper Ridge as that used by the writer. He used 66-ft. uncompensated grades, and got a wonderful amount of rise and fall into his line between Copper Ridge and Knoxville, about 130 ft. of it being between Copper Ridge summit and Beaver Creek.

At the crossings of the parallel valleys of Bull Run and Beaver Creek by the Southern Railway line, the latter valley is the higher by some 140 ft.; and the writer decided to cut through Copper Ridge, by a tunnel if necessary, at the level of the upper valley, to save all unnecessary rise and fall. It was found on examination that the two valleys, though one was so much higher than the other, had practically the same rate of fall, some 30 ft. in the 7 miles below the Southern Railway crossing, so that the point, *P* (see Figs. 2 and 3), in the open valley of Beaver Creek, was 140 ft. above the country at the mouth of Bull Run. At the point, *P*, however, where the stream turned and ran directly toward the only available gap in Copper Ridge, the valley began to descend much more rapidly down to the level of the Clinch. It was just possible, on the original 1% grade (see Plate XXI), to start the grade at the mouth of Bull Run and make the elevation of the upper valley by the time the summit of the ridge was reached at the gap. A cut, 68 ft. deep, containing 186 000 cu. yds. of clay, chert, and rock excavation, was necessary in cutting through the ridge. Supporting ground on the side of the ridge was found from the ridge summit to the point, *P*, for a level grade, so the road reached *P* without any unnecessary rise and fall. A water gap in the desired direction led through Beaver Ridge. This gap drained a portion of the open valley (Hines's) between Beaver Ridge and Black Oak Ridge into Beaver Creek, and the

PLATE XXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LII, No. 977.
TAYLOR ON RAILROAD LOCATION.





KNOXVILLE, LA FOLLETTE AND JELLICO R. R.

LOCATION THROUGH COOPER RIDGE
FROM
CLINCH RIVER TO BEAVER VALLEY

Scale of Feet
0 1000 2000 3000 4000

Contour intervals 20 feet

Dotted line in plan is location on 1.00% grade
Solid line in plan is location on 0.65% grade

three parallel valleys thus made a succession of steppes which were used to support the gradient from the Clinch to the summit of Black Oak Ridge at the lowest available gap. The alignment was excellent, and the country easy from the point, *P*, to the summit of Black Oak Ridge, and from there the road descended in fairly easy country to Knoxville.

There were, in all, five well-equipped engineer parties between Jellico and Knoxville, and the chiefs of parties were cautioned to use as light grades and curves as they reasonably could at all times.

Just here, at the risk of reiterating much that has already been written, the writer wishes to emphasize the necessity, in making such surveys as this:

(1) Of not using at any point any more difficult gradients nor any stiffer curves than the country actually requires;

(2) Of completing a condensed profile of the whole line as soon as the surveys are connected throughout;

(3) Of compensating for curvature the prevailing long grades of less rate than the maximum; and

(4) Of making a diligent study of the whole country, with a view of selecting a route with a minimum of adverse grade.

Some of the locating engineers were sent back over their work several times to see if they could not get through with lighter grades and curves, even though the grades and degree of curve used had been less than the maximum, and more than one of them made very material reductions in the grades which had been used, as well as in the degree of curve, without increasing the cost of the work materially.

When the condensed profile was made, after the lines were tied up, it was shown that there were only three points where it had been necessary to use 53-ft. grades against south-bound trains, *viz.:* the ascent to the Cumberland-Clinch divide, 8 miles in length; the ascent to Black Oak Ridge No. 2, 2¾ miles in length, and the ascent to Copper Ridge summit, 3 miles in length. Up to the time of the completion of this condensed profile, none of the higher officers of the road had hoped to get any better ruling grades through this country than 1% compensated. When the writer decided tentatively on the 1% grades, the consulting engineer of the road said to him, "If

you can make it from La Follette to Knoxville on one-foot grades you will be doing mighty well." No other road crossing these mountains anywhere in this section of country had secured anything like such a favorable gradient as this. Lines crossing these mountains, as far south as Birmingham, where the elevation of the Appalachian province is so much less, such as the Georgia Pacific line of the Southern Railway from Birmingham to Atlanta; and the Central of Georgia, Birmingham to Opelika, built in the Eighties, had used 66-ft. grades.

Except the three long grades mentioned, the heaviest south-bound grade was 0.65%; and the advisability of reducing these three 53-ft. grades to 34-ft. grades was at once suggested. What was the reduction worth and what would it cost?

The Cost of the Change.—It was plainly out of the question to reduce the ascent to the Cumberland-Clinch Divide to this gradient. It was too long, and involved construction which was too expensive. If the road was to be operated on the lower grade, it was plain that a helper engine must be used on this grade, and, therefore, the cost of maintaining a helper engine would be a legitimate charge against the change to 34-ft. grades. Assuming that the traffic would be proportioned as below, and that the schedule could be arranged so that one engine crew could do the helper service, the annual cost of this service would be about as follows:

Interest on $14 000, cost of helper engine, at 4%.....	$560
Twelve months' wages of crew, at $312.....	3 744
32 544 engine-miles, at a cost of 17.3 cents per mile, for repairs, fuel, water, stores and roundhousemen	5 630
Cost of 32 544 engine-miles to maintenance of way and structures, at $\left(22\frac{1}{2}\% \text{ of } \frac{\$1.08}{2}\right) = 12$ cents.....	3 905
Total annual cost of helper-engine service.....	$13 839

In the foregoing estimate, the cost of engine repairs, fuel, etc., is taken from the report of the Pennsylvania Railroad Company for 1902. It is assumed that the engine does half as much damage to the track as the train, the cost per train-mile being $1.08 and the cost to

maintenance of way and structures being 22½% per train-mile. We assume four round trips of 24 miles each per day, from the head of the Narrows to the end of the siding south of the divide, half the trains not running on Sunday.

Then, if the road's capital could be acquired at 4% interest, the cost of establishing and maintaining the helper service would be $345 975.

It was easily ascertained that by making a 700-ft. tunnel at Black Oak Ridge No. 2 the grade there could be reduced to 34 ft. without lengthening the line and without any more expensive construction, except the cost of the 700-ft. tunnel; and, further, the tunnel would save 50 ft., vertical, of the adverse grade.

Plate XXI is a map and profile of the old and new line at Copper Ridge. By dropping the grade line about 50 ft. at the summit and making a 2 170-ft. tunnel, and by taking advantage of the sharp fall in Beaver Creek Valley from the point, *P*, toward the gap, the 34-ft. grade could be used even here without lengthening the line and without increasing the cost, except by a portion of the cost of the construction of the tunnel. In fact, the lighter grade threw the line off from the upper cliffs of the ridge in certain places down on to the talus slope on smoother ground, with the result that a very much better line, both in alignment and first cost (except for the tunnel), was obtained. In the revision surveys here a maximum curvature of 68 was substituted for 10 degrees. This change had the effect of placing the tunnel of the summit of the ridge on the grade up to Beaver Creek Valley, and there was elevation enough to spare to permit of dropping the grade to 23 ft. per mile through the tunnel, thus compensating for wet rails by more than 0.2 per cent. Thus the cost of the change at these two points was not in excess of the cost of the other line by anything like the full cost of the tunnels. The construction contracts which had been made would warrant the belief that the tunnels could be constructed at a much lower figure, but, inasmuch as these tunnels had to be constructed in part through very treacherous clays, it will be supposed that they would cost $90 per linear foot. Not considering the 50 ft. of adverse grade, saved at Black Oak Ridge No. 2, the additional cost of the two tunnels would be about as follows:

700-ft. tunnel at Black Oak Ridge No. 2, at $90		$63 000
2 170-ft. tunnel at Copper Ridge, at $90		195 300
Total		$258 300
Less cost of summit cut at Copper Ridge:		
93 000 cu. yds., earth excavation, at 20 cents.	$18 600	
49 000 cu. yds., chert excavation, at 28 cents.	13 720	
44 000 cu. yds., rock excavation, at 70 cents.	30 800	
		63 120
Additional cost of tunnels		$195 180

The approaches to the tunnel at Black Oak Ridge No. 2 were very nearly as costly as the original summit cut. There was another expense which constituted a legitimate charge against the construction of the tunnels. Up to the time of the contemplated change, the longest tunnel on the road was in the mountain district, in good material, and was less than 900 ft. long. It was certain that the construction of these tunnels would take much longer than the remainder of the road; therefore, the interest on the money invested in construction, including the cost of the valuable property purchased in Knoxville for terminals, from the time that the other work could be completed until the tunnels could be completed, constituted a proper charge against the proposed change. Taking the amount expended in other construction, etc., as $3 500 000, with interest at 4%, and supposing that the tunnels delayed the opening of the road for traffic one year; then the total cost of changing to the lighter grade for south-bound trains would be as follows:

Establishing helper-engine service on the Cumberland-Clinch Divide	$345 975
Additional construction cost of two tunnels	195 180
Interest on $3 500 000 for one year at 4%, on account of delay	140 000
Total cost of change to lower grade	$681 155

The Value of the Change.—The value of such changes is difficult to estimate with accuracy, even on an operated road on which the traffic is known. It was reasonably certain that the road would not be in operation many years, if operated on 53-ft. grades, before as many as

ten trains per day each way would be required to do its business. Considering the traffic the road would have, probably six of these trains south-bound would be fully loaded. Four fully loaded trains per day, on a 34-ft. ruling grade, could just about do the work of six fully loaded trains per day, on 53-ft. grades, using engines and cars of the same class. Since the north-bound traffic would include a large quota of empty coal cars, and since the same engine that pulled, say, twenty-three loaded 40-ton cars southward against the 34-ft. grades could pull forty-nine empties back against the 53-ft. grades as far as La Follette and forty-four empties back, even against the 61-ft. grade, it was probable that eight trains per day each way on the lesser grade would do the work of ten trains per day each way on the higher grade. Operating expenses vary directly as the train-mileage, and the cost per train-mile on the Louisville and Nashville for the year ending June 30th, 1902, is given in "Statistics of Railways" by the Interstate Commerce Commission as $1.08. However, it is a fact that the operation of such heavy trains as these costs more than the general average on a large system. The cost per train-mile on the Duluth and Iron Range and the Duluth, Mesaba and Northern, two roads engaged almost exclusively in traffic similar to that which these omitted trains would represent, is given, by the same authority for the same year, as $1.94 and $2.61, respectively. Probably it would be safe to estimate the train-miles saved at $1.50, but the more conservative figure will be taken. Suppose that half these trains run on Sunday.

Then the annual saving from Saxton to Knoxville would be $1.08 x 4 x 79 x 339 = $116 000, nearly, and the value of this annual saving in operation, with interest at 4%, would be	$2 900 000
The cost of the change, as above	681 155
The amount gained by the change	$2 218 845

However, since helper-engine service could be established and maintained also at Black Oak and Copper Ridges, at a probable cost of $227 000 at each place, the engine mileage at either point not being more than half of that at the Cumberland-Clinch divide, the utmost amount it would have been economical to spend on the change at these two points was about $454 000. Thus it was more economical

to build the tunnels by the amount of $454 000 less $195 180 + 140 000, or $118 820. As soon as the President of the Louisville and Nashville Railroad understood the matter fully he authorized the change, without hesitation.

When the order was made to reduce the south-bound grades to 34 ft., there were some long minor grades on which the curvature had not been compensated, so that the total resistance was above that of the new ruling grade. As the construction had barely begun on most of the line, the corrections were inexpensive; but, if the road had already been in operation, the failure to compensate the prevailing minor grades would probably have resulted in the adoption of a higher ruling grade than that which was applicable to the country. Even if the heaviest grades had not been reduced at the time, with the probable heavy traffic in prospect, it was almost certain that sooner or later the heaviest grades would have been reduced or operated by helper engines, so that it is clear that the heavier minor grades should have been compensated at the outset.

It was also now contemplated to reduce the south-bound grades back on the operated road, between Saxton and Corbin, and also on the Atlanta, Knoxville and Northern for some distance below Knoxville, to the same rate. Thus the plan became a part of a more comprehensive scheme.

Now, it is true that, under ordinary conditions, if the road were laid out contemplating the use of a helper at any point, this adopted ratio of 1% and 0.65% for the helper grade to the ordinary is uneconomical; but, if the helper engine is to be of the same class as the regular engine, the helper grade, ordinarily, could be as heavy as 1.4%, if the grade for the regular engine was 0.65%. But there were very good reasons why this 1% grade against south-bound trains was not altered:

(1) The contract for the construction of the road through the mountain district had been let for some months, and the work was well under way before this change to 34-ft. grades was contemplated.

(2) The 1.4% grade would not have shortened the line at all, and the valley into which that grade would necessarily have descended was so narrow and rugged that there would have been little saving over the 1% grade.

(3) On account of the peculiar conditions under which the road

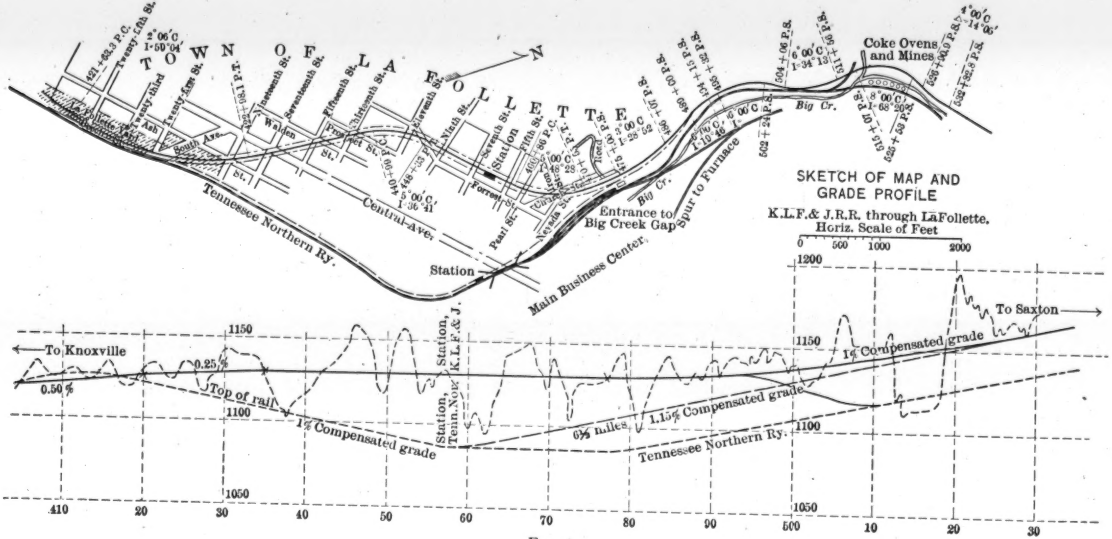

TOWN OF LA FOLLETTE
Tennessee Northern Ry.
Central Ave.
Station
Main Business Center.
Entrance to Big Creek Gap
Spur to Furnace
Big Cr.
Coke Ovens and Mines
SKETCH OF MAP AND GRADE PROFILE
K.L.F.& J.R.R. through LaFollette.
Horiz. Scale of Feet
0 500 1000 2000
To Knoxville
To Saxton
0.25 %
0.50 %
Top of rail
1% Compensated grade
1.15% Compensated grade
6⅝ miles
Station, Tenn. Nor.
Station, K.L.F. & J.
Tennessee Northern Ry.
1200
1150
1100
1050
410 20 30 40 50 60 70 80 90 500 10 20 30

FIG. 4.

would probably be operated, it was in no wise certain that the 1% grade would be used as a helper grade. But even if it were, and the grade revision was carried back or forward on the old road far enough to make up a complete freight division, there would be many a train which would not require the service of the helper on a 1% grade, and yet certainly would on a 1.4% grade.

The writer does not pretend that these adopted grades of 53 ft. and 34 ft. south-bound, and 53 ft. and 61 ft. north-bound, were studied out and adjusted accurately to the needs of the future traffic. The probable future traffic was considered, but the grades adopted were simply the best that the country afforded at a reasonable expenditure of from $40 000 to $60 000 per mile, below sub-grade, each of them being fixed by long, continuous ascents—to alter materially any one of which for the better could be done only at an expenditure not warranted by the anticipated traffic.

On the supposition that the plan of reducing south-bound grades on the old road was not carried out, the scheme of operation, as affected by these grades, without using a helper engine, could be outlined roughly as follows:

It was certain at the outset that a spur line would branch off at the south end of the Narrows to the excellent, but undeveloped, coal fields of the Clear Fork and Laurel Fork Valleys. It was also certain that a spur line would branch off in the neighborhood of Clinton to the coal fields in the vicinity of Oliver Springs. Thus, in operation, freight trains out of Knoxville would set off empties along the route until they were lightened sufficiently to pull up the 61-ft. grade north of La Follette. Going south, trains would leave the head of the Narrows with only such loads as they could pull on the 53-ft. grades, and would pick up enough loads at La Follette and other points on the route to load them fully into Knoxville.

The writer has been so impressed with the number of instances he has met in his practice in which engineers have lost sight of the important fourth principle, mentioned on page 477, that a special instance, illustrating how some expensive and unnecessary rise and fall was avoided, will be cited.

The survey for this road was commenced, under other engineers, at La Follette, at the station of the Tennessee Northern, and proceeded northward. The idea in mind was to parallel the Tennessee Northern through the town (see Fig. 4), and to use the station, already

built, for both roads. The same parties controlled the Tennessee Northern, all industries, and most of the land near La Follette, and it was considered by them of very great importance to their plan of development that, through the town, the old road should be paralleled by the new. The 6½-mile, 61-ft. grade, from the Cumberland-Clinch divide did not "touch bottom" until just before the station was reached. The profile shows "the hole" in which the station would have been if the original location had been retained. It happened that it was some months after the original line from La Follette northward was run before the writer sent an engineer to La Follette to begin the survey for the extension of the line toward Knoxville, and the idea that the location of the road through the town was fixed as above had pretty well crystallized in the minds of all concerned. This engineer's instructions were to start at the summit south of La Follette (at Station 410, which was an objective point) with the grade elevation of the Tennessee Northern in the summit, and find supporting ground back through the town for a level grade, extending it back into the gap to an intersection with the long 61-ft. grade. The map and profile show the line that was secured. It was about as cheap to construct as the other, gave less curvature, and was some 700 ft. shorter. It placed the station about 300 yds. further from the center of the town than the other station would have been, but good connection with the tracks leading to all the La Follette industries could be had at the summit mentioned and at the point, *F*. This point was just south of the intersection of all tracks on the Tennessee Northern leading to all the coke ovens and coal mines, and was just north of all tracks leading to the three crushers, brick-yards and furnace; and the only land available for yards for the new road, in the vicinity, was at and near this summit at Station 410.

There was vigorous opposition to the new location through the town, however, but the common-sense view of the matter prevailed in the end, and the road was built, as it should have been, on the upper grade line.

The comparative advantage in operation of this upper line was so palpable that it was never attempted to calculate how much more economical it was than the other line. To have put the station for this important town in such a place as was contemplated, with 43 ft., vertical, of unnecessary maximum grade immediately on each side of it, would have been bad engineering, even if the traffic from this point

alone was to be considered; but to put a station on a through line in such a place, at which all heavy trains would probably have to stop, when it could be avoided at any reasonable outlay, would have been the height of folly.

After all the contracts for the construction of the main line and the Clear Fork Branch were let, it was found that it required very expensive construction to connect the Oliver Springs line, either at Lee's Ford or just on the north side of the Clinch, 2 miles below Clinton, which were regarded as the only available points. The proposed routes united near Dosset. To reach Dosset from Lee's Ford required 3 miles of road, a bridge over the Clinch, and a tunnel through Black Oak Ridge No. 2. To reach Dosset from the second point required 5 miles of very heavy work and a tunnel through the same ridge. Either plan of building the branch line, then, would necessitate tunneling this ridge twice, once for the main line north of Clinton and once for the branch line southwest of Clinton. Since it was a fixed fact that the branch line would be constructed, it was suggested that it might be better to turn the main line southwest at a point 3 miles south of Coal Creek, take it through Dosset and make the point of junction for the branch line at Dosset. This plan would save the construction of one tunnel, and either one bridge over the Clinch and the construction and operation of 3 miles of road or the construction and operation of 5 miles of road. But it would miss the town of Clinton. Examination showed that, when the construction of both the main line and the branch line was considered as a whole, the proposed change was advantageous in point of alignment, distance, and economy. The gradients of the main line were not benefited materially by the change, but those of the branch line were benefited very much. To tunnel the ridge on the branch line, with the gradients adopted for it, would have required 2 500 ft. of tunnel, but the gradients now decided upon for the main line required 3 520 ft. of tunnel. Thus, by saving the 700-ft. tunnel and the 2 500-ft. tunnel, and making the 3 520-ft. tunnel, there was 320 ft. more of tunnel to construct, but the cost of the expensive approaches to one of the tunnels was saved.

The town of Clinton was already well provided with railway facilities, and it had not the population nor any industry to give it claim for consideration against such advantages. The last census

showed that it was decreasing in population. Therefore the line was changed to pass through Dosset; and the further advantage was secured that, against south-bound trains, the two principal branch lines have the same limiting gradients as the main line. Thus the road is laid out somewhat on the same principle as that by which a stream departs from its direct course to meet its principal tributaries.

The road, as located finally, is 78.8 miles in length, from Saxton to Asylum Street in Knoxville, making the length of road exceed the straight-line distance 55%, instead of 34%, as by the Knoxville and Ohio Branch of the Southern between the same points.

The roughest country encountered was from the head of the Narrows to the Cumberland-Clinch Divide, and here 10° curves were used. Quite expensive construction was necessary on the south side of this divide as far as La Follette, and here a maximum of 8° curves was used. Rough country was also encountered in the 6 miles north of Coal Creek, and on the ascent through Copper Ridge. The sharpest curves used at the last two points were 6°; the remainder of the road was located on light curves, so that the points where curvature would reduce high speeds materially were bunched. From Saxton to La Follette there are ninety-two curves, the total angle turned being 3 717°, or 145° of curvature per mile. From La Follette to Knoxville there are one hundred and nineteen curves, with a total central angle of 3 692°, or 69° of curvature per mile. The minimum tangent was 300 ft., and all curves above 2° were spiraled. The Holbrook spiral, with three different rates of spiraling, was used. For curves under 5°, a spiral increasing 1° in 60 ft. was used (called a 60-ft. spiral). For curves above 5° and under 7°, a 30-ft. spiral was used, and for curves of from 7 to 10°, a 24-ft. spiral was used. It will be noticed that the rate of spiraling is changed sharply in passing from curves of about 4° to those of lesser radius. The reason for this was that curves of 4° and less, with 60-ft. spirals, were intended to be used in open country, and the curves of higher degree, with the 30-ft. spirals or less, were intended to be used only in country where it was necessary to use curvature of such high degree as would necessarily limit the speed of fast trains. The curves were located originally without spirals, but the resident engineers put in the spirals just before staking out the work, so that the road was constructed on the spiraled alignment. It will be noticed that these spirals would all fit in, usually with some distance to spare, on the 300-ft. tangents.

The grade breaks were rounded off by vertical parabolic curves, changing the rate of grade 0.2 ft. per 100-ft. station at summits, and 0.1 ft. per 100-ft. station in sags.

On the final location adopted, there were seven tunnels, in the 79 miles of road, aggregating nearly 10 000 ft. in length. One of these tunnels in the mountain district was located partly on a 10° curve, so that the spiral approach curve, with its varying curvature and rail superelevation, came within the tunnel. The rule used for increasing the tunnel section, so as to give at every point practically the same clearance as on tangents, for an 80-ft. Pullman or dining car, was:

"Widen the tangent section $\frac{1}{16}$ in. per degree, both on the inside and on the outside of the curve. In addition, widen the section on the inside of the curve $2\frac{1}{16}$ ins. per each inch of superelevation."

This particular tunnel was in hard sandstone not requiring lining. The tangent section was rectangular from subgrade to 17 ft. above (16 x 17), with curves of 5-ft. radius in the upper corners. The rule gives a slight excess of clearance on the outside of the curve, which, to some extent, allows for the compression and swing on the car springs.

At the time of writing this paper (December, 1903), practically all the grading, except that in the Knoxville yards, is completed. Several of the tunnels are completed, but the 3 520-ft. tunnel at Dosset cannot be completed, probably, before some time in the summer of 1904. The main line is being built with permanent structures, no timber bridges or bridge supports being used, except four wooden trestles which are to be filled in, eventually, with steam shovel and train.

Of openings on the main line, there are eleven concrete and stone masonry arches of from 12 to 26 ft. span. There are thirty steel bridges, on masonry piers and abutments, and two viaducts of 1 420 ft., total length. These steel bridges consist of three through spans, one of 150 ft. and two of 200 ft., and sixty-seven plate girders of the following kinds, writing their lengths to the nearest 5 ft.: One 40-ft. through, one 45-ft. through, one 80-ft. through, and one 45-ft. double-track deck; and single-track deck girders of the following lengths and number: Two 14-ft., six 30-ft., six 40-ft., eleven 45-ft., one 50-ft., nine 60-ft., twenty-one 70-ft., one 90-ft., and one 120-ft. In these structures there are 43 529 cu. yds. of first- and second- class bridge and arch masonry. In addition, there are 37 248 cu. yds. of culvert masonry.

The total cost of construction of the main line ready for operation, exclusive of rolling stock and equipment, is now estimated to be $5 450 000, or a little in excess of $69 000 per mile. The cost of the Oliver Springs Branch is estimated to be $300 000, and of the Clear Fork Branch $93 000.

As will be inferred from the foregoing discussion, in the lay-out of the road, far more weight was given to gradient and the securing of traffic than to curvature and distance. If economy in operation is in any wise proportional to the smoothness of the grades, it is an economically located road, considering the country. In seeking a measure of grade smoothness for such a purpose, the smoothness of a gradient should be taken to vary inversely as the total number of feet, vertical, ascended and descended on maximum, or at least on expensive, grades; but, taking the smoothness to vary inversely as the total number of feet ascended and descended, some interesting comparisons may be shown.

From the crossing of the Clinch, at Clinton, to Knoxville, on the Knoxville and Ohio Branch of the Southern, the total rise is 641 ft. and the total fall 601 ft., or a total average rise and fall per mile of 59.2 ft. From the crossing of the Clinch, on this line, to Knoxville, there are 290 ft. of rise and 264 ft. of fall, making a total average rise and fall per mile of 21.5 ft. Measured as above, the smoothness of this new line exceeds the smoothness of the old line, in the same country, by 173 per cent.

In this respect, the smoothness of this road compares favorably with the smoothness of some prairie roads. The average total rise and fall per mile from Saxton to Knoxville is 26.2 ft. The average total rise and fall per mile of one of the principal roads from Kansas City to Chicago, for the first 100 miles out of Kansas City, across the Missouri prairies, is 37.2 ft. per mile. Thus, measuring grade smoothness by this standard, the smoothness of this mountain road, crossing the direction of the prevailing ridges and ranges, exceeds the smoothness of this prairie road by 42 per cent.

DISCUSSION.

Mr. Low. EMILE LOW, M. AM. SOC. C. E. (by letter).—The writer spent about nine years in the section of country adjacent to that traversed by the Knoxville, La Follette and Jellico Railroad, five years of which were in the service of the Norfolk and Western Railroad, and the other four years with the Mathieson Alkali Works, of Saltville, Va.

In discussing Mr. Taylor's interesting paper, the writer desires especially to confine himself to a description of the Clinch Valley Division of the Norfolk and Western Railroad, with which he was very prominently identified.

During 1886, it was considered advantageous by the representatives of the Louisville and Nashville and the Norfolk and Western Railroads to enter into closer communication. Both systems controlled a considerable mileage of railroad, but were somewhat widely separated, the nearest points being Pineville, Ky., on the former, and Graham, Va., on the latter line, the distance separating them being about 200 miles.

The central point was in the vicinity of Big Stone Gap, Va., which was at first selected as the junction point. The junction was subsequently changed to Norton, Va.

As it was considered expedient to hasten the construction of the "missing link" with all possible speed, the authorities of the Norfolk and Western Railroad caused a hasty reconnaissance of their portion of the line to be made late in 1886. The report of this reconnaissance stated that apparently a very favorable line, as to grades and alignment, could be secured, the line after leaving Graham to follow the Bluestone River to its head, a distance of about 10 miles, with a maximum grade of 53 ft. per mile, then, crossing the divide near Springville, Va., to the Valley of the Clinch River, which was to be followed to the mouth of Guest River, a distance of about 90 miles, with maximum grades of 53 ft. per mile, then ascending Guest River, to Norton, Va. (then known as Princes Flats), a further distance of about 20 miles, with maximum grades not exceeding those of other parts of the line. Shortly after the reconnaissance was made, several survey parties were placed in the field. One, starting at Graham, covered the eastern end of the line and, after reaching the Clinch River, kept to its north bank. This preliminary survey disclosed a decided peculiarity of the Clinch River, in the excessive sinuosity of the stream, which for long stretches formed a series of escalops. The same conditions existed at the western end, especially near the mouth of Guest River, and along the latter stream.

The section traversed by the Clinch Valley Division of the Norfolk and Western Railroad lies on the western slopeof the Appalachians,

Mr. Low.

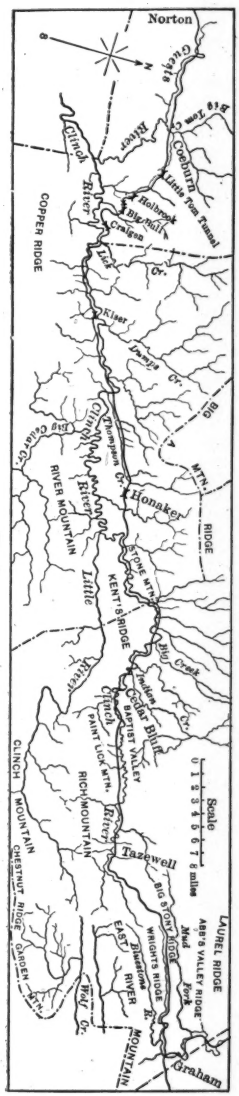
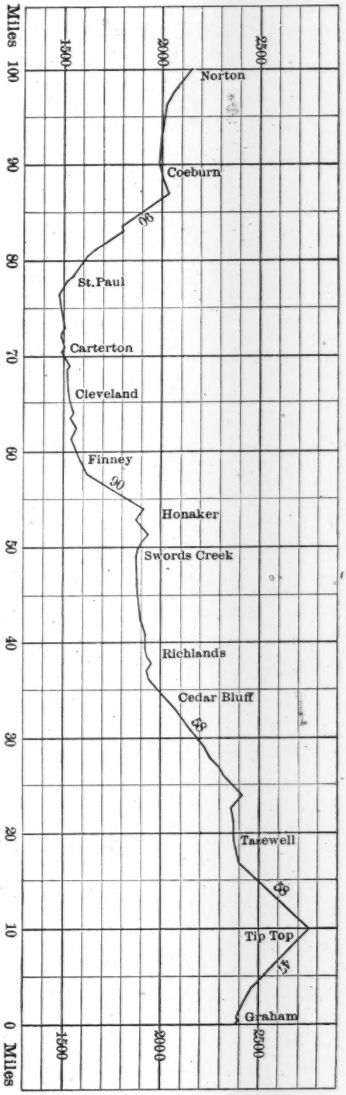


Fig. 5.

NORFOLK & WESTERN RAILROAD

CLINCH VALLEY DIVISION



Mr. Low. which, in this region, are made up of many local ranges, the general trend of which is about S. 60° W. The Clinch River flows mainly between the Clinch Mountain on the south and Sandy Ridge and Cumberland Mountains on the north, the latter being also the general southern limits of the West Virginia and Eastern Kentucky coal fields. The railroad line just skirts the edge of these coal measures, which are reached by short branch lines.

It was at this particular stage that Mr. Taylor's Rule 4 came prominently into play. This rule reads:

"(4) Of making a diligent study of the whole country, with a view of selecting a route with a minimum of adverse grade."

Owing to the extremely adverse topographical conditions in the valleys, it became absolutely imperative to seek for the most favorable ground, no extremely radical departure from the selected route being possible.

A new survey developed the existence of a somewhat lower summit at the head of Wright Valley Creek, a tributary of Bluestone River, and the line was transferred to the valley of the former stream, a maximum non-compensating ascending grade of 47 ft. per mile having been adopted.

As at the head of the Bluestone River, the North Fork of the Clinch River heads against the source of Wright Valley Creek, and the line follows this fork to the main river, with a maximum compensated descending grade of 58 ft. per mile.

From the foot of this grade, the fall, for 6 miles, was comparatively light, being about 12 ft. per mile. The river then began to fall more rapidly, at the same time impinging upon the southern slope of the Baptist Valley Ridge and gradually cutting through this ridge and its continuation, Kent's Ridge, at Cedar Bluff, Va. An examination of the profile (Fig. 5) shows that the line leaves the Clinch River at Mile 22.5, passing over what is locally known as Young's Summit. Here occurs the first adverse grade (after passing Tip Top) in a generally descending grade of 66 miles. This diversion from the river was justified on account of the excessive cost of construction between Pisgah and Pounding Mill had the river been followed more closely.

Between Pounding Mill and Cedar Bluff, the Clinch River is extremely sinuous, the distance between two points by river being more than twice that in a straight line. As there was no possible alternate route here, the river was followed, entailing extremely heavy work. This section of the line is shown in detail in Fig. 6.

After leaving Cedar Bluff the fall of the river diminishes, being, for 15 miles, to Sword Creek, about 140 ft., or less than 10 ft. per mile. Although the course of the river is somewhat sinuous in places, there are no steep banks or bluffs. At Richlands there is one adverse grade, which could have been abolished at a comparatively slight cost.

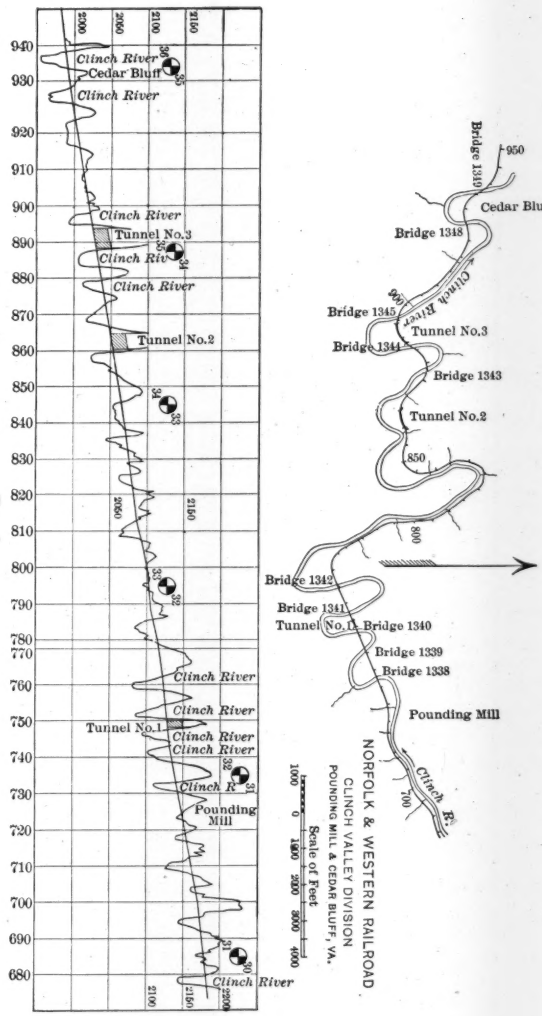

Clinch River
Cedar Bluff
Clinch River
Clinch River
Tunnel No.3
Clinch River
Clinch River
Tunnel No.2
Clinch River
Clinch River
Tunnel No.1
Clinch River
Clinch River
Clinch R.
Pounding Mill
Clinch River
Bridge 1349
950
Cedar Bluff
Bridge 1348
Clinch River
Bridge 1345
Tunnel No.3
Bridge 1344
Bridge 1343
Tunnel No.2
850
800
Bridge 1342
Bridge 1341
Tunnel No. 1
Bridge 1340
Bridge 1339
Bridge 1338
Pounding Mill
Clinch R.
700
NORFOLK & WESTERN RAILROAD
CLINCH VALLEY DIVISION
POUNDING MILL & CEDAR BLUFF, VA.
Scale of Feet
1000 0 1000 2000 3000 4000

Fig. 6.

Mr. Low. It was the original intention, as stated previously, to follow the river between Sword Creek and the mouth of Thompson Creek, and the first survey was made on this course. An examination of the general map, however, will show the tortuous windings of the stream between the points named.

North of Honaker there is a high spur known locally as "Big A" Mountain, the slope of which is toward the south. Through the base of this mountain the river has cut its way, leaving on the south side a range, variously known as River Mountain and also as Elk Garden Ridge.

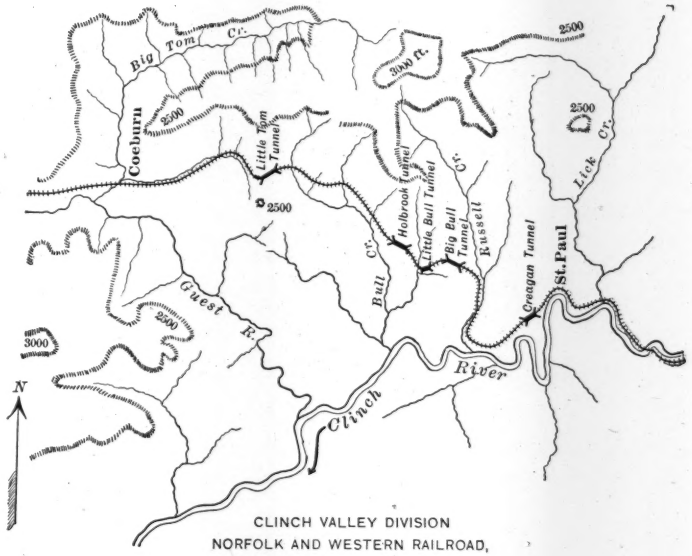
As the cost of the river line seemed prohibitive, an alternate line, involving less expense, was sought. It was found that a stream called Thompson Creek paralleled the river to the north for about 8 miles. The headwaters of this stream could also be reached from Sword Creek by a cross-country line, which utilized largely a branch of Lewis Creek.

Surveys of this cut-off developed the fact that maximum grades of 53 ft. per mile could be obtained, but in order to secure them a tunnel 3 000 ft. long would be required. As this was considered too expensive a feature, the maximum grade was increased to 90 ft. per mile, the modification resulting in reducing the length of the tunnel to 900 ft. at the Honaker Divide. This change of line resulted in two adverse grades of 53 ft. per mile between Sword Creek and Honaker, and greatly increased the ruling grade to the eastward. From the mouth of Thompson Creek to St. Paul the line again followed the river. Between these places some adverse grades occur at points where the line cuts across large bends in the river, these grades having been adopted in order to minimize the excavation.

Perhaps the most difficult portion of the line was the location between St. Paul and Coeburn. The enormous expense of building a line up Guest River precluded this route at the beginning. Another cross-country line was then adopted. This line, as finally developed, left the valley of Clinch River, at Lick Creek, above St. Paul, gradually ascended the ridge forming the north bank of the river until the top was reached, near Creagan Tunnel, and then following Russell Creek, passing through Big and Little Bull Tunnels to the waters of Bull Run, all of which streams are tributaries of Clinch River. In order to reach the main valley of Bull Run the line passed through a spur by means of the Holbrook Tunnel. Another tunnel, Little Tom, occurred at the head of Bull Run and gave access to the waters of Little Tom Creek, a tributary of Guest River. The total elevation overcome was 556 ft. in a distance of 10 miles, corresponding to an average grade of 56 ft. per mile, although the maximum compensated grade used was 90 ft. per mile.

From Little Tom Tunnel the line descends to Coeburn, with a

Mr. Low.



CLINCH VALLEY DIVISION
NORFOLK AND WESTERN RAILROAD,
BETWEEN ST. PAUL AND COEBURN, VA.

Scale of Miles

0 1 2 3 4 5 6

FIG. 7.

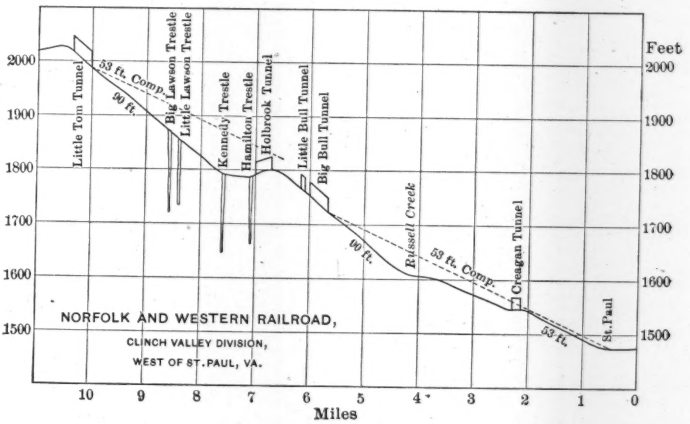


FIG. 8.

Mr. Low. maximum grade of 30 ft. per mile, following the valley of Little Tom Creek, until Guest River is reached, which stream is then ascended to Norton, the maximum grade being 53 ft. per mile.

Referring again to the location between St. Paul and Little Tom Tunnel (near Coeburn), it may be stated that, in order to overcome the rise with a compensated grade of 53 ft. per mile, a grade line more than 11½ miles long would have been required, making necessary a development of 1½ miles or more. The foot of the grade could not well have been shifted to the east of St. Paul, as this is the proposed crossing of the Ohio River and Charleston Railroad (better known as the 3 C's Railroad), where a joint yard is proposed.

In order that the heavy nature of the work on this portion of the line may be more fully understood, a few of its characteristics are given in Table No. 1.

Not counting the smaller trestles, there are four high ones, approximating 150 ft. in height.

TABLE No. 1.

Name of tunnel.	Mile.	Elevation.	Length.
			Feet.
Creagan	79	1 547–1 547	905
Big Bull	82	1 725–1 754	1 695
Little Bull	82	1 761–1 766	327
Holbrook	83	1 801–1 992	1 566
Little Tom	87	1 989–2 022	1 925

To the initiated, an inspection of the profile west of St. Paul presents some vagaries, especially the two well-defined dips in a long ascending grade line. The first is caused by the crossing of Russell Creek, where the present crossing is 42 ft. high. A 53-ft. compensated grade would have made necessary a crossing 80 ft. high, and would have intersected the present grade line at the east end of Big Bull Tunnel; or, to intersect the present grade at the west portal would have required a development of about ¼ mile. This lighter grade, undoubtedly, would have increased the cost of construction between Creagan and Big Bull Tunnels, and the amount of traffic alone would determine whether such an additional outlay was justifiable.

Between Big Bull and Little Tom Tunnels, the situation is more serious. In order to introduce a compensated grade of 53 ft. per mile, there would have been required a development of 1 mile of line, or a tunnel about 1 mile long at Little Tom Divide. The first would have increased the height of the high trestles to 200 ft., or more, and also would have added some other highly interesting engineering features. Had this railroad been constructed on a 53-ft. grade, it

would unquestionably have been one of the most spectacular on this continent. Mr. Low.

Before closing, the writer would like to add a few remarks in regard to the proper method of conducting railroad locations. It is the practice to cut up a proposed line of railroad into sections and allot one of these to each survey party. This is good as far as it goes, but the trouble is that each party is generally allowed to follow its own bent, in fact, to roam at its own sweet will, instead of filling in a part of a harmonious whole. Bad locations also result from improper organization, an insufficient number of members in the parties, delegating to the chiefs of parties such work as sitting up all night in a tent plotting the day's work, when such duties ought to be performed by draftsmen, especially provided for in suitable quarters, as well as in daylight.

With proper methods, surveys can be kept plotted up to date, and the locating engineer can thus have a broad view of the ground he is covering, and also an intelligent idea of where to run additional lines, when needed to cover doubtful points. In addition, copies of all notes, including transit, level and topographical notes, should be sent (or mailed) to headquarters, at frequent intervals, to be worked up at once.

This enables the chief engineer to see what is being done in the field, and also keeps him in close touch with all his assistants. Thus he can give such orders as to changes and improvements in the whole line as will result in producing a perfect design.

WILLIAM P. WATSON, M. AM. SOC. C. E. (by letter).—This paper has been read with much pleasure. The author is correct in the statement: Mr. W. P. Watson.

"The economic questions which determined that the whole project of construction was advisable may be of more importance than the most intricate and learned calculation upon the strength or efficiency of special structural parts."

By the former, the economic success or failure of the enterprise as a whole is determined, while the failure of the latter affects, generally, only one part, and often a very small part, of the whole enterprise, and can be remedied easily by a more skilful and complete structure. For this reason, a skilful and scientific location of any given line of road is very rarely appreciated. As formerly stated, the work of the locating engineer may determine the success or failure of any given enterprise.

Of course, the determination to construct a line between two termini is the province of the executive branch of the road, but, even for this purpose, who can be a better adviser than the skilled engineer? After this has been determined, there come the minor economic questions of the selection of the route, wherein must be considered: First,

Mr. W. P. Watson.

the best route for traffic and the probable volume thereof; and secondly, the various questions as to the proper gradients, distance, rise and fall, and curvature, their degree of importance being in the order named. The first affects the revenue of the road, and the last four affect the yearly operating expenses, or the profits of the enterprise. They are as legitimate and proper subjects of consideration by the locating engineer as is the first cost of the construction; for they are taken care of by the annual operating expenses and the latter by the annual interest, or fixed charges; and yet what great sums are spent on railway construction without even considering any of these questions, except, probably, gradients. The writer has known chief engineers, who, after the ruling gradient had been decided upon (probably by the executive department, or management, from what it wanted and not from a scientific study of the subject), appeared to attach no importance to the other questions, indeed, had no thought or opinion upon them; and, when a comparison between any two lines was made, almost invariably took the line that showed the cheaper construction, no matter if obtained by almost any excess of distance, rise and fall, or curvature. And yet such men are entrusted with works of great magnitude, amounting to thousands and millions of dollars. It is only the abounding prosperity of the nation, and not the skill of the engineer, which, when such methods are allowed, makes an enterprise a success instead of a failure. It is therefore refreshing to see a location in which such questions have been considered.

The writer has given the paper only a hasty examination, but notes the following points:

Adverse Grade.—It would seem that the author has given an undue and disproportionate weight to the subject of adverse grades, and has fallen into the common error of adding up all the rises and falls, no matter how small, for any two compared lines, and giving the preference to the one having the greater aggregate total. This may be very fallacious, as that showing the lesser may have all its rises and falls in two or three long stretches of 10 000 ft. or more, and be very objectionable, while the other may have an undulating grade with numerous short rises and falls, of less than, say, 30 or 40 ft., so arranged that traffic can be handled as cheaply as on a level grade, upon the theory of "momentum or velocity grades" which is illustrated as follows:

Theory of Momentum or Velocity Grades.—A constant power (the locomotive) is applied to overcome the train resistance, then the undulations, or various rises and falls, are overcome by the momentum, or stored energy, which is increased, or stored up, on the descents and given off on the ascents, as illustrated in Fig. 9. This is what engine men mean by the expression "taking a run at a hill."

Fig. 9 represents the profile of an imaginary line whereon a locomo-

TABLE No. 2.

Mile.	Elevation (or lift), in feet.	Difference in elevation of sags and summits, to be added or subtracted.	Equivalent lift, in feet.	Equivalent speed, in miles per hour.
100.....	0 (It is assumed that at Mile 100 the train has a speed of 30 miles per hour)			30
102.2.....	95.8	+ 32*	127.8	60
104.4.....	127.8 (lift due to 60-mile speed)	− (95.8 + 24)	8.0	15
107.2.....	8.0 (lift due to 15-mile speed)	+ 24.0 + 64.8	96.8	52
110.....	96.8 (lift due to 52-mile speed)	− (64.8 + 20.0)	12.0	18
114.....	12.0 (lift due to 18-mile speed)	+ 20.0 + 62.0	94.0	51 +
115.1.....	94.0 (lift due to 51-mile speed)	− (62.0)	32.0	30

*Original potential lift, due to a speed of 30 miles per hour.

tive is supposed to be able to haul a train on a level grade at 30 miles per hour and to exert the same tension upon the draw-bars up and down hill, the alignment being arranged so as to allow of very fast speeds. The calculation of the different speeds is illustrated in Table No. 2.

A train in motion at 30 miles per hour has a potential lift of 32 ft., vertically.

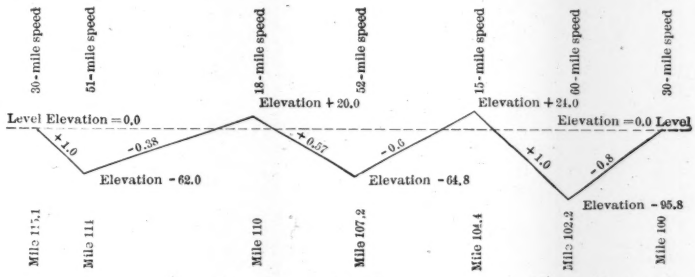


FIG. 9.

Thus the table shows that at Mile 115.1 the speed is 30 miles per hour, or the original starting velocity.

Is it not evident that a road constructed on such a profile can be worked as cheaply as an equivalent level road (except for the practically unmeasurable wear and tear on track and rolling stock due to varying speeds), and with actually no more application of power, provided the velocities can be maintained safely?

The writer would not advise carrying this to the extent suggested by Wellington, that is, using grades greater than the maximum, in these sags and rises, for, although this is correct, theoretically, it pre-supposes always an unobstructed track. While this may generally

Mr. W. P. Watson.

be so, it may not always. Suppose the grades on each side of the sag at Mile 102.2 were beyond the maximum. Is it not apparent that any obstruction necessitating a stop then would stall the train and impede, or tie up, the traffic for the entire division, invalidating all its fine-spun theories and elaborate calculations? A slippery rail near the summit might have the same effect, for, though this might not interfere with the potential lift, it would interfere with the constant force, the locomotive, because, with freights especially, the ability to start a given load up any grade limits the amount of that load, and, if it were not for the elasticity in the springs of the couplers, it is doubtful whether an engine could start a load which it could readily pull.

This would indicate that these sags could be made 80 or 90 ft., but the line must be located for its weakest member, which would be a freight of 40 or 50 cars, with speeds of from 15 to 30, or 25 to 40, miles per hour.

With speeds of from 15 to 30 miles per hour the safe sag would be

$$(3.0^2 \times 3.55) - (1.5^2 \times 3.55) = 24 \text{ ft.}$$

With speeds of from 25 to 40 miles per hour, the safe sag would be

$$(4.0^2 \times 3.55) - (2.5^2 \times 3.55) = 35 \text{ ft.}$$

Therefore, a line located with an undulating grade, containing sags not exceeding 30 ft., or on special occasions 40 ft., would be within safe limits, provided the ruling grade were not exceeded.

The foregoing calculations are dependent on the ability to maintain fast speeds. Therefore, anything which would interfere with this should be avoided. A sharp curve at the bottom of a sag, say at Mile 102.2 in Fig. 9, would not be permissible. Such a curve half way down would not be as objectionable, or could be introduced at a summit without harm, if properly compensated.

Distance.—The author has hardly given due weight to the question of distance, and the writer is not sure that the advantages attained are sufficient to compensate for the lengthening of the line about 11 miles more than that of the Southern. The operation of an increased distance of this extent will probably cost an additional 50 cents per train-mile. The cost for 10 trains daily each way, at $1.08 per train-mile (the cost on the Louisville and Nashville System), is calculated as follows:

$50 \times 365 \times \$1.08 \times 2 \times 10$ trains = $4 042 per year added to general operating expenses for one train-mile, and, for 11 miles, this makes $44 462, which, capitalized at 4% per year, gives $1 111 550.

Thus it would be justifiable to expend on first construction more than $1 000 000 to save this distance.

Now, from an examination of Fig. 2, it looks as if this excess distance was lost between Knoxville and La Follette, and it also appears that a 1% line could be obtained. Suppose such a line to have been

Mr. W. P. Watson.

located, following, in the main, the Southern from Knoxville to the Clinch, then, which seems to be the natural route, up the Clinch to Big Creek and up latter to La Follette. It is generally cheaper and better to keep the railroad line as near the natural drainage ways as practicable. This would also have had the advantages of opening a new and unoccupied territory. The selection of the line through La Follette and from there to Jellico seems to have been eminently wise and correct.

This supposed line between Knoxville and the Clinch would have about the same rises and falls as the Southern, the extreme elevation of any point being about the same as that of the Knoxville, La Follette and Jellico Railroad between the same points. But the former has the additional rise and fall, out of and into the valleys of Beaver and Bull Run Creeks, which amount to about 125 ft. for the former and 225 ft. for the latter, or a total of about 350 ft. Now, in continuous stretches of 100 ft. and more, such rises and falls add to the yearly operating expenses $1.85 per foot of rise and fall per train-mile, or, for this line, with the assumption of 10 trains per day, each way, and $1.08 per train-mile, this would be

$$\$1.85 \times 10 \times 2 \times 1.08 = \$39.96,$$

which, capitalized at 4% per year = $999. That is, it would be justifiable to spend about $1 000 per foot to eliminate the rises and falls. The comparison, with the increase in yearly operating expenses, for the two items of distance and rise and fall, capitalized, is as follows:

The Present Knoxville, La Follette and Jellico Railroad Location.

To 10* miles distance (increased) at $100 000 per mile	$1 000 000
First cost of excess 10 miles, say $60 000	600 000
Total excess of present location	$1 600 000

Supposed Location.

To 350 ft. of rise and fall at $1 000	350 000
Total in favor of supposed location	$1 250 000

This is upon a 1% (or 53 ft. per mile) ruling gradient.

But the present location is adapted for a ruling gradient of 35 ft. per mile on all south-bound traffic, and the cost of putting this supposed line upon this basis should be deducted. Now, it seems highly probable that by constructing heavy tunnels, say two, 4 000 ft. long, through the summits of Copper and Chestnut Ridges, this could be

* Only 10 miles' increase is charged, as 1 mile may be lost in developing the supposed line to a 1% grade.

Mr. W. P. Watson.

accomplished. It would look as if it would be better to have a helper for the Bull Run Valley, the grade for which could be 1.55%, or 82 ft. per mile, two engines hauling upon this what one would haul on a 0.65% grade. One engine would be sufficient, as the distance would be only 4 miles, and only south-bound trains would have to be helped. Again, probably 5 miles of extra line would have to be built for the Oliver Springs Branch, and would probably cost $30 000 per mile. There would also be two larger tunnels. The helper engine and the 1.55% grade would probably bring the question of rise and fall under the rule of momentum grades, and eliminate the question from the comparison entirely. The mile of distance allowed for development would be reduced probably as much as ½ mile. The statement would then compare about as follows:

To excess cost of present location (as before).....			$1 250 000
For reducing the proposed 1% line to a 0.65% line for south-bound business:			
2 tunnels (8 000 ft.) at $100.....		$800 000	
1 helper service (the author's cost is assumed for this, though it is thought to be high)		350 000	
Branch to Oliver Springs, 5 miles at $30 000.		150 000	
		$1 300 000	
But from this is to be deducted the following:			
350 ft. of rise and fall saved, at $1 000.....	$350 000		
½ mile of distance, including cost of construction.....	80 000		
		430 000	
			870 000
Balance in favor of the proposed line.....			$380 000

If such a change could have been made, on the foregoing basis, it should have been done, as this is bound to be a link in the main line, and subject to severe competition. There is also the probability of the traffic increasing, thus making the comparison continually more favorable than that given.

Of course, the foregoing is merely a supposititious case, and might be wholly impractical of realization in the field, but, at least, it shows the danger of ignoring such an important economic question as 11 miles of increased distance.

Mr. W. P. Watson.

TABLE No. 3.—ESTIMATED PROBABLE COST OF DOUBLING THE ENGINE TONNAGE TO HANDLE THE SAME TRAFFIC, UPON 1 MILE OF RAILROAD, DUE SOLELY TO AN INCREASE IN THE MAXIMUM OR RULING GRADIENT.

SUBDIVISIONS.	PERCENTAGE.		ADDED COST.	
		Affected.	Details.	Total.
MAINTENANCE OF WAY AND STRUCTURES.				
Repairs of roadway	0.11732	80	0.09386	
Renewals of rails	0.02581	80	0.02065	
Renewals of ties	0.04154	80	0.03323	
Repairs to bridges and culverts	0.02000	80	0.01600	
Repairs and renewals of fences, road crossings, signs and cattle guards	0.00757	.....	.....	
Repairs and renewals of buildings and fixtures.	0.01582	.....	.....	
Repairs and renewals of docks and wharves	0.00005	.....	.....	
Repairs and renewals of telegraph	0.00261	.....	.....	
Stationery and printing	0.00011	.....	.....	
Other expenses	0.00092	.....	.....	
Totals	0.23175	.....	.....	0.16374
MAINTENANCE OF EQUIPMENT.				
Superintendence	0.00768	.....	.....	
Repairs and renewals of locomotives	0.07372	90	0.06635	
Repairs and renewals of passenger cars	0.02318	05	0.00116	
Repairs and renewals of freight cars	0.05718	10	0.00572	
Repairs and renewals of work cars	0.00248	.....	.....	
Repairs and renewals of shop machinery and tools	0.01040	.....	.....	
Stationery and printing	0.00036	.....	.....	
Other expenses	0.00396	.....	.....	
Totals	0.17896	.....	.....	0.07623
CONDUCTING TRANSPORTATION.				
Superintendence	0.00700	.....	.....	
Engine and round-house men	0.09375	80	0.07500	
Fuel for locomotives	.09337	75	0.07003	
Water supply for locomotives	0.00873	70	0.00611	
Oil, tallow and waste for locomotives	0.00429	70	0.00300	
Other supplies	0.00125	70	0.00087	
Train service	0.06545	80	0.05236	
Train supplies and expenses	0.01609	80	0.01287	
Switchmen, flagmen and watchmen	0.03070	50	0.01535	
Telegraph expenses	0.02341	.....	.....	
Station service	0.06902	.....	.....	
Station supplies	0.00347	.....	.....	
Car mileage balance	0.04516	.....	.....	
Loss and damage	0.01250	.....	.....	
Injuries to persons	0.00743	.....	.....	
Advertising	0.00378	.....	.....	
Outside agencies	0.02840	.....	.....	
Commissions	0.00048	.....	.....	
Rents of buildings and other property	0.00126	.....	.....	
Stationery and printing	0.00456	.....	.....	
Other expenses	0.04010	.....	.....	
Totals	0.56020	.....	.....	0.23559
GENERAL EXPENSES.				
Salaries of general officers, insurance, law, etc.	0.02909	.....	.....	.....
Estimating the extra engine to cost $12 500, and capitalizing at 4% per annum, the interest charges would be $500; and estimating the daily engine run to be 110 miles, or 40 000 miles per annum, the cost per mile is	.....	.....	.....	0.01250
Total	.....	.....	.....	0.48806

Mr. W. P. Watson.

Ruling Gradient.—In the author's treatment of this point he is slightly in error. In the first place, it is improper to assume that trains are not fully loaded, for it is the duty of the management to see that the power is used, up to its full safe capacity, thus obtaining the greatest economy. If the grades or traffic in one direction are different from those of the other, as they are, almost always, this must be made up by the size or number of trains.

Secondly, while operating expenses may vary directly as the train-mileage, they do not vary as the number of trains. This is shown in Table No. 3.

Thirdly, it seems to the writer that $2 900 000, the result of the author's calculation for the saving by the change in the ruling gradient, is fully one-half too large, and that the method shown in Table No. 3, which is practically Wellington's method, is the better. The writer has taken the annual operating sheet of the Missouri Pacific Railway for 1902, which will serve as a standard, and has reduced the various items to percentages, the gross total being unity.

The value or percentage in Table No. 3 (0.48806) is for one train one way, but if a change is made in the maximum, or ruling, gradient, it affects the load on the entire division, which, for convenience, may be called 100 miles long.

Therefore, to double the engine-tonnage to haul the same traffic, due solely to a change in the ruling gradient, on a division 100 miles long, will cost $48.806 per 100 train-miles per day, or $48.806 × 365 = $17 894.20 per annum; or, as the train-mile on the Louisville and Nashville costs $1.08, this will be $17 894.20 × 1.08 = $19 325.70 for increase of 1 engine-mile, or 100% per year per daily train one way only.

Now, an engine will pull 73% as much on a 1% grade as upon a 0.65% grade, or, to haul as much, will increase its mileage $\frac{1.00 - 0.73}{0.73} = 0.369$ per cent. But an increase of 1 engine-mile, or 100% per year per daily train for a 100-mile division, is $19 325.70, and for a 79-mile division and 10 trains daily, one way only, this change will be worth $19 325.70 × 0.369 × 0.79 × 10 = $55 511 saving in the operating expenses, which, capitalized at 4% per annum, gives a capitalized saving of $1 387 775 by changing the ruling gradient from 1% to 0.65 per cent. But even this saving would fully warrant the change, which, therefore, was a wise and proper one.

It is manifest from an examination of Table No. 3, showing the operating expenses of the Missouri Pacific Railway, that there are many items therein which are not affected by an increased train service, as, for instance, general expenses, superintendence, etc., etc.

Vertical Curves.—For a link in a main line, when fast speeds are

desired and expected, a gradation of 0.05 ft. per station (100 ft.) in sags and 0.10 ft. at summits makes a better and more easy transition than those mentioned by the author. Some recommend even a flatter curvature, but, with the present general use of springs in couplers, 0.05 and 0.10 ft. are sufficient. Mr. W. P. Watson.

Transition Spirals.—A spiral increasing 1° in 60 ft. is too short for a line of this character, as it is suitable only for a speed of about 36 miles per hour.

The writer believes that very seldom has a line received such thought and study regarding the main questions of location—the economic questions—and the author deserves great credit for laying it before the Society so fully. The writer regrets that time prevents him from going into the subject more thoroughly.

E. J. BEARD, M. AM. SOC. C. E. (by letter).—This paper is dedicated primarily to the young and inexperienced, and, therefore, it is important to call attention to some statements and computations wherein the author, in setting forth some of the considerations which determined important steps, has evidently committed errors, the pointing out of which, it is hoped, will prevent some of the dedicatees from falling into the same mistake. Mr. Beard.

It is not true, as the author states and applies, that the "operating expenses vary directly as the train-mileage," for only a part of the total operating expenses is affected by the addition or deduction of one or more trains. It is not necessary to enter into history to verify this statement, as all railroad engineers (and, for that matter, all operating officials) should be thoroughly familiar with the principles underlying, and so admirably set forth in, the work of the late A. M. Wellington, M. Am. Soc. C. E. While the percentage cost of each item of expense to the cost of a train-mile, as contained in Wellington's "Economic Location," is not strictly correct at this date (due to change in conditions), his tabulations can be quickly brought up to date by the application to them of the data found in detail in the reports of the various State Railway Commissioners, the annual reports of various railroads, or, sufficient for all practical purposes, from the work quoted by the author.*

Wellington states that some 50% of the cost per train-mile is affected by an increase or decrease in the number of trains. While there is some variation, under ordinary conditions, in the cost of each item since Wellington wrote, his figure, 50%, is not, at this date, very far wrong when the change (*i. e.*, the increase or reduction in train-miles) is considerable, but when such a small reduction in the number of trains is accomplished, as by the author on the road in question, it is doubtful if the change would amount to more than 35

* "Statistics of Railways in the United States," compiled by the Interstate Commerce Commission.

Mr. Beard. or 40% of the average cost ($1.08) per train-mile on the Louisville and Nashville Railroad.

The statement that he is conservative in adopting this cost per train-mile, and in justifying the statement by referring to the cost per train-mile on the Duluth and Iron Range and the Duluth, Mesaba and Northern Railroads, is hardly to be considered true after looking into the operating conditions shown by the statistics of these roads, coupled with a belief in the very different operating conditions of each. From the United States Statistics of Railways, before referred to, the percentage cost per train-mile of the four grand divisions, referred to the total cost of a train-mile, is as shown in Table No. 4.

TABLE No. 4.

	L. & N.	D. & I. R.	D., M. & N.
	Percentage.	Percentage.	Percentage.
Maintenance of way	21	27	35
Equipment	22	27	25
Transportation	52	42	34
General	5	4	6
Totals	100	100	100

This table shows that the excessive cost of maintenance of way and equipment (notably the former), compared with that of transportation, is excessive as compare with that shown for the Louisville and Nashville. Although the writer is unfamiliar with any of the roads named, it would seem evident that climatic and other conditions on the Michigan roads, compared with those on the Louisville and Nashville, are the cause. There is nothing in the paper to indicate excessive maintenance expense, and, no doubt, it will at least average with that of the system of which it is a part, which, it is ventured to say, varies little, mile for mile, in the items of expense affected by trains or engine, when the number of trains is equal.

There are seemingly several reasons why the average train-mile cost on the road in question, if it had been built on a 1% maximum, should not be any higher than the general average of the Louisville and Nashville. One is that there are many items included in the cost per train-mile (which is merely the unit of the grand total of the operating expenses of a great system, and includes many items, such as general officers, general office forces and other expenses), which will not in any wise be affected by the addition of this small mileage to the total of the road. In other words, it is not reasonable to assume that, when the cost of operation per train-mile on 3 000 miles of road is

Mr. Beard.

$1.08, the addition of 79 miles, over which is to pass the same average train-tonnage per annum, will increase the annual operating expenses by the annual train-miles on 79 miles at $1.08 per train-mile. What has been said indicates the wide reasoning necessary to be conservative, and to be so that something less than $1.08 would be likely to be in order. However, to show more fully the wrong results arrived at by the author, that figure will be used later.

The author states that it was "in no wise certain that the 1% grade would be used as a helper grade." How, then, is he justified in recommending the expense of reducing the other two 1% grades to 0.65 per cent? The truth, no doubt, is that, whatever the number of trains, they will (as far as it is safe for the engineer to go) practically all arrive at the foot of the hill loaded to require a helper, as the operating department will hold for full train loads, unless the unbalancing of traffic should occasionally require return of power to the other end—an exigency unsafe to figure on—and the only proper thing to do is to figure on pushing all the trains estimated to be required for the traffic to be handled. Again, why run the helper engine 4 miles more than the length of the helper hill? It should not, and is not likely to, be done, as it would give 16 or 17 miles, say (with properly located sidings), as the length of the helper's round trip, instead of 24 miles, and for 8 trains per day, or, with half that number on Sundays, 46 104 engine-miles per annum. The author also wrongly states that 4 trains on the 0.65% grade will handle the same tonnage as 6 trains on the 1% grade, representing a decrease of $33\frac{1}{3}\%$ in engine- or train- mileage, when the relation between the grades in this respect is a reduction of 27 per cent. While this is true, probably a reduction to 8 trains should be estimated on instead of the exact ratio, 7.3.

From the foregoing data is obtained the annual saving in operating expenses due to the reduction in the number of trains, as follows:

20% of 20 trains (10 each way per day) $\times$ 35% of $1.08 $\times$ 79 miles $\times$ 39 days per year = $40 492.87.

This sum, capitalized at 4%, gives but $1 012 322, a sum vastly less than that arrived at by the author.

In determining the effect the operation of the helper engine will have on transportation, maintenance of way, and equipment expenses, it would have been better to have obtained the data from the statistics of the Louisville and Nashville instead of taking those of the Pennsylvania Company, where conditions of operation are distinctly different, and adaptable to but few roads in this country. In lieu of nothing better than the "Statistics of Railways," those can be used, which, no doubt, conform closely enough for the purpose to the conditions that would be encountered. Obtaining therefrom the following, as the

Mr. Beard. percentage per train-mile of operating items affected for maintenance, equipment and transportation (except wages) gives:

Repairs and renewals of locomotives....	7.3	per cent.
Superintendence and general expenses..	0.6	"
Fuel....	10.8	"
Water.....	0.65	"
Oil and waste.....	0.35	"
Other supplies.....	0.2	"
Telegraphing and despatching	1.1	"
Total.....	21.0	per cent.

It is no doubt true that the expense of these items, in the case of a helper engine, would vary somewhat from the general average of a road engine of the same pattern, but, in view of the severe and erratic service required of the helper, it would probably be more than for a road engine.

The assumption that half of the maintenance-of-way expenses is due to engines alone is, to say the least, a crude method, if one stops to think of how the introduction of a helper would affect in any manner whatever most of the items making up the total of such expenses, except as to roadway, rails and ties. Probably something like one-half of that proportion of the rail wear due to trains is due to the engines alone, but that ratio does not, as assumed by the author, apply to any of the other items, and the most that can be charged to maintenance of way is something like the following:

Renewals of rails.....	1	cent,
Repairs to roadway.....	3	cents,
Ties.....	0.6	cent,
Switches and sidings.....	0.4	"
Miscellaneous.....	1	"
Total.....	6.0	cents,

or about one-half the author's estimate. The coincidence of the total of transportation, and maintenance of way and equipment about equalling the author's, of course, has no meaning. His erroneous reasoning would give an error of $\pm$ 25 cents per train-mile, or about 50% in such a case as the Duluth, Mesaba and Northern. Summing up these items gives, as the annual cost of one helper engine with a double crew, the following:

Interest on $14 000, cost of helper engine, at 4 per cent.....	$560
Annual wages of crew	3 744
46 104 train-miles at 27 cents.	12 448
Total.....	$16 752

Mr. Beard.

This is somewhat nearer the annual cost of a helper engine, which, by the writer, is almost invariably estimated at $18 000 per annum, and, in view of the many non-estimable objections to the use of a helper, is none too large. To draw attention to what some of these non-estimable objections might consist of, take this 8-mile helper hill. It will be found in practice that 8 freight trains per day each way is about the limit of one helper's capacity on such a hill, and its use will tend to delay trains at the foot of the grade, delay meetings at other places, and generally interfere with the even operation of the road, in comparison with what it would be if no helper were required. All these delays mean expense—fuel, wages and other concomitant items —and probably $20 000 would not be too high to estimate as the annual cost of this pusher; but this is not intended to decry the use of pushers. From these figures, the estimated cost of change, to lower the ruling grade, would be as follows:

Annual cost of a helper engine on the Cumberland-Clinch Divide, $18 000 capitalized at 4 per cent	$450 000
Construction cost of two tunnels	195 180
Interest on construction for one year, having no earning power during the construction of tunnels	140 000
Total cost of change to lower grade	$785 180

There are other increased expenses, due to a reduction in ruling grade, to add to this. One, often overlooked in estimating the saving in operating cost accomplished by the reduction in train-miles for the same tonnage traffic obtained by the reduction of ruling grades, is that this reduction increases the cost of operation per train over every grade plane the rate of which is less than the ruling grade. Stated in another way, the value of a foot of rise and fall varies with the ruling grade. An engine loaded for the higher ruling grade can proceed on any less grade at a speed equal to that attainable if loaded for the lower ruling grade, with the consumption of less fuel; or, in another way, like engines will require and consume more time over a given division after its ruling grades are reduced to a lower maximum than before the change, when loaded for the respective ruling grade, resulting in increased expense for all items affected by time. The result of disregarding this fact is that often the advantages and saving in operating expenses estimated on by the engineers are not realized, and, consequently, the expected decrease in cost per ton-mile is not realized, the operating department finding it necessary to decrease the engine rating below that indicated by the differences in ruling grades, in order to get trains over the division within reasonable

Mr. Beard. time and with economy. Take the line under discussion: It will require from 15 to 20% more time for a train, having an engine loaded for a 0.65% maximum grade, to go from Saxton to Knoxville than it would for the same engine loaded for a 1% grade. This, probably, would not affect train wages, in view of the shortness of the division, if operated as one division; and whether or not it is to be operated that way is understood as yet to be unsettled. But, no matter how it is operated, time affects fuel, water, oil, etc., all of which will be increased from 15 to 20%, and some other items not necessary to enter into here to affect the reasoning. If the cost of fuel, oil and water is, as has been assumed, 11.8%, this gives an expense here (due to this condition) of 20% of 11.8% of \$1.08 × 16 trains × 79 miles × 339 working days = \$10 922.

\$10 922, capitalized at 4%. gives.....	\$273 038
Increased cost (before obtained).....	785 180
Total.....	\$1 058 218

This amount wipes out entirely the saving expected by the reduction of ruling grade. While but an approximation, this is not far from the truth, and, without a doubt, the President of the Louisville and Nashville Railroad certainly does not yet understand.

It will not be amiss to warn the inexperienced not to take literally all the author's remarks about smoothness of grade lines, by which he awkwardly expresses absence of rise and fall, which, though an important matter, is minor to that of ruling grades. Its value is readily estimated, and, to avoid a few feet more or less on minor grades, will not warrant a very great expense. Neither should he take literally the implication that it is easier to obtain the best across Missouri's northwest prairies than through Tennessee's mountains. Many who have had experience in both kinds of country will bear out the remark that the contrary is true. Titanic work is not the embodiment of good engineering, and the 40% more rise and fall on the Missouri road, considered with the conditions of years ago under which it was built, together with its cost (perhaps \$15 000 to \$20 000 per mile, at present prices), as against \$69 000 per mile for the Tennessee road, does not, by any means, indicate better engineering on the latter. The elimination of the 10 or 11 ft. more rise and fall per mile would not warrant, with like traffic, an additional expenditure exceeding probably \$2 000 per mile, giving a cost of from \$17 000 to \$22 000 per mile, as against \$69 000. An examination of a map of the country crossed by the Chicago-Kansas City lines shows their general direction to be across the drainage, and each valley is from 250 to 350 ft. below the saddles of the ridges, thus approaching those of the author's road.

Mr. W. Watson. WALTER WATSON, ASSOC. M. AM. SOC. C. E. (by letter).—While the general rules to be followed in making a location given by Mr.

Taylor are vital and quite full, the writer would extend them somewhat by making the first one read: Mr. W. Watson

(1) Of not using at any point more difficult gradients, nor any greater distance, nor any more curvature, nor stiffer curves than the country actually requires.

The writer would also suggest the desirability of adding another rule, *viz.*:

(5) Of occupying, at all strategic points, as fully as possible (without otherwise injuring the location), all the available ground.

Many engineers do not fully appreciate the value of distance and curvature saved, nor do some seem to appreciate the fact that it is not always the line that shows the least construction cost per mile that is the cheapest, even to construct. As an illustration of the effect of distance and curvature the following experience is cited:

The writer recently had occasion to revise a projected location, made previous to his engagement, through a very crooked valley with high bluffs on each side. These bluffs placed seemingly insurmountable obstacles in the way of any location, except one which followed the general windings of the valley, and, even then, quite heavy work was required. At one place, covering about 1½ miles of the original location, it was found, however, that by introducing two tunnels (of about 550 and 260 ft. in length) a line could be located, with the same gradient, with lighter curves, with about 320° less curvature and about 2 170 ft. shorter. Although the construction of the new line was more expensive per mile, it was actually about $9 000 cheaper than a line on the original location would have been, besides being much less expensive to operate.

Without in any way seeming to criticise the location shown on Plate XXI, for which it is presumed there were most excellent reasons, the writer would like to ask if it was on account of lack of sustaining ground from the Bull Run side of Copper Ridge toward Lee's Ford (territory outside of the limits of the plan on Plate XXI), that the long detour around Copper Ridge was used, rather than a line up Beaver Creek from Open Valley, near the point *P*, and then passing directly under Copper Ridge by a tunnel to the Bull Run slope of the ridge, or whether there were other considerations entering into the problem? Plate XXI would indicate that possibly such a line, with the same rise and fall and gradient as the line located, and from 1½ to 2 miles shorter, but requiring an indicated tunnel about 2 500 ft. long, compared with one 2 170 ft. long on the line as located, might have been used, provided suitable sustaining ground could have been obtained toward Lee's Ford.

In regard to the suggested rule (5), it may be stated that there are often cases in difficult country where the line can be properly located, so that it would be extremely difficult for a rival company to construct

Mr. W. Watson.

a line through the same pass or valley, while a location, equally good in other respects, could be made, which would leave room for a rival to pass, and thus the situation would not be fully controlled. The complete control of such a pass, or valley, might add very materially to the value of the property.

In further discussion of the general subject of railroad location, the writer would place what he considers the four chief elements entering into the problem in the following order:

First.—The reduction of the ruling grade to as low a point as practicable for each engine division, considered separately.

Second.—The elimination of as great an amount of distance as is consistent with true economy.

Third.—The reduction of the total amount of curvature as much as possible, and using as light curves as the nature of the ground will permit.

Fourth.—The matter of rise and fall.

The writer would determine the value of any reduction of the ruling grade of an engine division (or the amount which one would be justified in spending to reduce the ruling grade) by ascertaining the number of daily trains each way that could be dispensed with in handling a given traffic on account of the reduction of grade. In this it is generally only necessary to consider the freight trains, as the number of passenger trains required is generally independent of the grade. Ordinarily, the only effect of grade on passenger trains is in the matter of speed, and not in the number of trains.

The cost of each additional train required (or the saving made by dispensing with a train) is not the average cost of operating all the freight trains on a division, as there are certain items (such as superintendence and general office expenses) which enter into this average cost which are not affected in any way by the number of trains, and others which are affected only partially.

The writer places the cost of each additional train at approximately 42.5% of the average cost per train. Therefore the capitalized value of the saving of one train a day each way would be (about)

$$0.425\,\frac{\text{Average cost per train-mile} \times \text{twice the length of the engine division} \times 365}{\text{Rate of interest.}}.$$

For example, assuming the length of an engine division to be 100 miles, the average cost per train-mile \$1.08, and the interest rate 4%, the expenditure allowable for reducing the ruling gradient sufficient to affect the saving of one train a day each way over the division would be

$$0.425\,\frac{\$1.08 \times 200 \times 365}{0.04} = \$837\,675.$$

If, in comparison of the cost of maintaining a pusher engine for service over a portion of the division with the cost of the reduction

of the ruling grade over the whole division, it appears that the pusher grade is the most desirable, this pusher grade should be so adjusted with reference to the lower ruling grade of the engine division as to utilize the full effective power of the pusher engine. This effective traction power is theoretically the full tractive power of the additional engine, but, for this service, it is not safe to assume more than 90% of the full tractive power of the extra as being available, as there appears to be a practical loss of some 10% of the tractive power of the engine when used in this way. Mr. W. Watson.

The pusher grades, when found advisable, should be grouped as much as possible, and placed preferably near one end of the engine division, and, if possible, located in such a manner that, if increase of traffic (or other considerations) should at any time make it advisable to reduce them to the ruling grade on other portions of the division, they may be so reduced without changing any more (or very little more) of the line than is actually occupied by these heavy grades.

Also, if it is found practicable to use a lighter ruling grade on one end of an engine division than on the other end of the same division, it is advisable to do so. For example, if it should be found that on 50 miles of a 100-mile division it was practicable to use a 0.35% grade, and on the other 50 miles it was found impracticable to use a ruling grade of less than 0.60%, it would be good practice to locate these two parts as if they were separate divisions, using the lowest ruling gradient practicable for each part, for, at some future time, connections may be made, or other conditions arise, which would make it advisable to operate this proposed 100-mile division as two divisions, or as parts of other divisions, of which one could have a lower ruling gradient than the other.

Except for the effect on speed, the writer can see no especial economy in reducing the ruling grade to less than 0.35%, as the starting resistance of a train on a level tangent about equals the rolling resistance at 10 or 12 miles an hour on this grade. In other words, an engine that can start a train on a level tangent can take it up a grade of 0.35% at 10 or 12 miles an hour. Also, when stations or sidings occur on a ruling grade (or one approximating a ruling grade), it is best, where possible, to reduce to a grade 0.35% less than the ruling grade for the length of the siding and for a train length on each side thereof.

All ruling grades should be compensated for curvature, and it is often in accordance with the best practice to compensate for curvature on other than ruling grades. The compensation on ruling grades should be from 0.03 to 0.04% per degree of curve.

At all breaks in grade easy vertical curves should be used.

When the traffic in opposite directions is not evenly balanced it is often advisable to use a heavier grade against the lighter traffic than

Mr. W. Watson.

against the heavier, but the grade against the lighter traffic should not be made as steep as to require more trains to move it than are required for the heavier traffic. In balancing the grades in the two directions, it should be borne in mind that there will probably be a greater proportion of empties to loads in the direction of the lighter traffic, and that, in proportion to the total weight of train, a train of empties presents a greater resistance than a train of loads. The grades should be adjusted accordingly.

The reduction of distance should be divided into three classes for the purpose of valuation:

First.—Those which are so short as not even to affect the wages of train crews.

Second.—Those which would affect the wages of train crews, but would not affect the number of sidings required.

Third.—Those which would affect the wages of train crews and also the number of sidings required.

The cost of each additional train-mile, on account of additional distance for each of these classes (as shown approximately by notes in the writer's pocket notebook) is: For the first class, 31.25% of the average cost per train-mile; for the second class, 45% of the average cost per train-mile; for the third class, 58.5% of the average cost per train-mile.

The cost of each additional degree of curvature per annum (calculated from known curve resistance, etc.), used by the writer in field comparison of lines, is 16% of the average cost per train-mile.

For example, assuming 5 trains a day each way; average cost per train-mile, $1.08; and rate of interest, 4%; the capitalized value of every degree of curvature saved would be

$$\frac{0.16 \times \$1.08 \times 10}{0.04} = \$43.20.$$

In considering rise and fall it is also necessary to divide into three classes:

First.—Those cases in which the rise and fall is so little that it is not necessary to shut off steam in descending (that is, when the working of the engine is not affected), or, in other words, when the rise and fall does not affect the "virtual profile."

This class will generally include all cases of rise and fall not exceeding from 25 to 30 ft. (1 ft. up and 1 ft. down being called a foot of rise and fall), but, in order to determine its limits exactly it would be necessary to construct a "virtual profile" for each separate class of travel (fast passenger, local passenger, fast freight and slow freight), as trains of some one or more classes might be affected, while the others would not be.

In making these "virtual profiles," the effect of stations, sidings, coaling stations, water tanks and all other features which might

require stops or low speed should be taken into account (as well as curves and the effect of their location on permissible speed). It should also be borne in mind, when making these "virtual profiles," that the train resistance is greater at high speeds than at low speeds, and that the effective tractive power (draw-bar pull) of the engine is less at high speeds than at low speeds. These considerations tend to make the dividing line between rise and fall which does not and that which does affect the "virtual profile" somewhat lower than it is often calculated as being. Mr. W. Watson.

Rise and fall which does not affect the "virtual profile" does not affect the cost of operating the line appreciably, and may be introduced quite freely. Even here, however, the ruling grade should be exceeded only in rare and exceptional cases, and then only for short distances. When one exceeds the ruling gradient in any way, one is treading on very dangerous ground.

Second.—Rise and fall which requires shutting off steam, but not the application of brakes, to keep proper control of the train.

The writer's pocket notes show that the additional cost per annum, for each daily train one way, of operating 1 ft. of that portion of this class which affects the "virtual profile" is nearly 46% of the average cost per train-mile, if on minor grades. If on ruling grades, this cost should be taken at about 86% (the additional allowance being made to cover injury to track and rolling stock on the ruling grade).

Third.—All cases of rise and fall which require both the shutting off of steam and the application of brakes to control the train properly.

In cases of this kind it will be found that the additional cost per annum of operating 1 ft. of rise and fall for each daily train one way is about 96% of the average cost per train-mile if on minor grades, or 136% if on ruling grades.

At the present time the writer has not access to his library or notes, other than those contained in his pocket notebook, and all the foregoing figures are taken from that book and are approximate only; but they are near enough to the exact values to serve for the comparison of the majority of alternate lines. If, after a comparison of lines by these approximations, there is the least doubt as to the proper line, they should be compared more carefully in the office.

Another consideration, which should be given weight in making a location, is the time that the several trains can make over the road, as time occupied in the moving of freight and passengers has a money value and sometimes a great value. However, as the time that can be made over a division is influenced, not only by the distance, grades, sharpness and location of curves (whether at the top, bottom or middle of a grade), but also by the amount and character of the traffic, the location of stations and sidings, length of sidings, location of

Mr. W. Watson.

water and fuel, location of signals and length of blocks (where the block system of operation is used) and, finally, by the "personal equation" of managers, superintendents, train masters and dispatchers, it appears to be a rather uncertain quantity, and one for which the writer knows of no method of calculating a value which would be of any service for general application, and can only say that it should be taken into consideration in making a location.

All curves should be located (or constructed) with easements.

Generally speaking, the best location (and construction) for a given traffic is that for which the sum of the interest on the capital invested and the expenditures for maintenance, renewals and operation is the least.

All the foregoing applies most particularly to the location of new main line for which sufficient capital is available to construct the "best" road to serve the traffic. If the available capital is not sufficient to construct the "best" road for the traffic, "the garment should be cut to the cloth," and a location made which can be operated economically and which will come as near as possible to the "best." At all places where departure is made from the ideal it should be located so that in the future the proper line can be built with as few changes and as little loss as possible. For a company which has not sufficient capital available to construct a road which can be operated so as to earn fixed charges, operating maintenance and renewal expenses, the writer does not think that any good location can be made. A location which will throw the company into the hands of a receiver is, generally speaking, not a good location, although, at the same time, it might be a most excellent location for a road with larger available capital.

Into the problem of the selection of the best line comes the consideration of reaching centers of traffic. It is a question sometimes as to whether the main line should be run to (or near to) these centers, whether they should be reached by spur or even loop tracks, or whether they should be neglected. The determination of these questions will depend largely on the quantity and character of the traffic expected for the main line and on the quantity and character of the traffic expected from and to each particular point.

For the location of branches, feeders and industrial spurs of all kinds, the problem of the proper location is apt to be quite different from that presented in the location of a main line. Here, the best location is often that which will afford the cheapest construction, and is not as dependent on the cost of operation as the chief profit derived from the traffic of the branch or feeder may be in the haul of the freight after it reaches the main line. (The branch, taken by itself, may be operated at a loss, and, at the same time, may be a great source of gain to the road operating it.) The best location for the branch may lie between that showing the smallest construction cost and that

which would be the best location for a main line, or may even be controlled almost entirely by the desirability of locating it so as to keep, as far as possible, a competitor from reaching a given point. Mr. W. Watson.

In making any location, for either main line, branch or change of old line, great weight should be given to the geological formation, the likelihood of the occurrence of slides and washes, and the influence of swampy or other treacherous ground. The likelihood of blockades from drifting snow or sand should be considered also.

Water and coaling stations should be located in the most favorable places for the starting (and stopping) of trains, and it will often be found that it is more economical to pipe water quite a considerable distance than to place the tank immediately at the source of water supply.

The problem of improving a line already constructed is different in many ways from that of locating a new line, because any radical changes of line or grade will necessitate the throwing away of line already constructed, with seasoned roadbed, and on which the strong and weak points of structures are known. Or it may be influenced even more largely by charter and right of way and other acquired rights than a new location would be. In the case of change of old line, however, the data for making comparisons of lines are more exact than with a new line in that the amount and character and the cost of handling the same are better known and the position of stations, sidings, etc., is fixed more definitely.

In changes of old line it will frequently develop that it is advisable to leave (or perhaps even to introduce) certain grades, curves or other features which would not be permissible in the location of a new line.

In order to make a good location, the engineer should have a good idea of the problems of construction and the means of solving them.

After all has been said, and the most careful comparisons and calculations have been made, no hard and fast rules can be laid down either for selecting the main points of route or in carrying out the details of location of either a new line or a change in an old line. The element of experience, natural bent and "personal equation" of the engineer (and his associates in other departments) will enter largely into the problem.

While all the matters touched on above and others not mentioned come into the province of the engineer for decision or advice, it occurs quite frequently that some of them are decided without consulting him. In such cases he should do the best he can with the conditions imposed, and come as near as possible to fitting his location to the traffic and finances of the company, under the circumstances.

WILLIAM G. RAYMOND, M. AM. SOC. C. E. (by letter).—This paper is of the greatest interest to all those having to do with railroad location. There are one or two points that will perhaps bear emphasis. The first Mr. Raymond.

Mr. Raymond. is the great value of the topographic charts of the United States Geological Survey. Time and again have these charts been used in projecting water-supply work. Time and again have they revealed reservoir sites and conduit routes which would have been entirely missed by the reconnoitering engineer. And, for the use of the railroad locating engineer, they are even more valuable, not only saving his time, but insuring, with the exercise of reasonable care and intelligence, the selection of the most feasible route between two termini by compelling the following of Wellington's precept, stated anew by Professor Taylor in his fourth emphatic suggestion, that the reconnaissance shall be of an area, not a line, and by furnishing a small-scale, bird's-eye view of the widest possible area in such a way that it may all be taken in at a glance, all seen at once, and be all at once before the engineer for study. The writer has used these charts for both railroad location and water-supply work, and has found them invaluable aids. The completion of the charting of the entire country cannot be accomplished too soon.

A second point is that the low grade line does not always mean the most expensive line even in first cost; that, in general, an economical location is one which adopts the lowest possible grades for the longest possible distances, and bunches the difficulties at a few points where they may be overcome most economically by heavy work or by pusher service.

A third point is the evidence furnished by this paper of the advance of railroad location from a trade to a profession; from the "rule-of-thumb" methods of many years ago to the scientific methods of to-day; and this without reflecting on those great men who, early in the history of railroad building, saw the true relation between location and operation. Moreover, the paper emphasizes the necessity for taking time and spending money in the design of the road, and by design is meant location, and for doing the work over, revising and re-revising, until the best that can be done by the united efforts of all the engineering talent engaged has been accomplished.

One suggestion: The paper professes to be written for the young men. Therefore, from the teacher's standpoint, should not the qualifying adjective "capitalized" be used before "cost" in the second paragraph on page 479?

One other suggestion: In determining the cost of pusher-engine service, only those items of expense which may be attributed to this particular service are used, a proceeding that seems rational; but, when it comes to a consideration of the saving due to the reduction in train-miles by the lower ruling grade, it is stated that "operating expenses vary directly as the train-mileage," the cost per train-mile on the Louisville and Nashville Railroad for the year 1902 is used in the calculation following, and the total saving is found by multiplying

the train-miles saved by this train-mile cost, $1.08. This, being the average cost of all trains, is, of course, as the author points out, not the proper cost to apply to the heavy freight trains, the ones affected by the grade changes considered, but is used for safety, the actual cost of trains not yet running being indeterminate. The author estimates that the average train-mile cost of these heavy trains may be as great as $1.50; but, whatever it is, will the saving in money, due to a saving of train-miles, equal the cost per train-mile multiplied by the train-miles saved? Manifestly, it will not, and a considerable part of the late Mr. Wellington's great work was devoted to showing that it will not be this quantity. Railroad bookkeeping may make it so appear, and an increased number of trains due to increased business does not always seem to lessen the train-mile cost, as it should, theoretically, but this is due largely to increased expenses warranted by the increased business, and is not at all due to the greater number of trains. In recent years train-mile cost has increased greatly, but this is not due to the running of more trains. When the business does not change, and a reduction in train-miles, due to a reduction in grades, or increased locomotive power, is secured, it is doubtful if more than half the average train-mile cost is saved for each train-mile saved. Just how much the saving is, no one knows. In the present case, if $1.50 per train-mile is a fair cost for the trains affected, and half of this can be saved with each train-mile saved, the annual saving will be $80 343, instead of $116 000, and the amount gained by the changed line will be $1 327 420, instead of $2 218 845. The reduction of nearly $900 000 in the capitalized saving in no wise changes the correctness of the final choice in this case, but it might well change it in some other case where the difference in values might not be so great. With an assumed train-mile cost of $1.50, it is entirely possible that each train-mile saved may mean $1.08 saved, but the evident purpose of the author was to use the whole train-mile cost, making only as much allowance as would insure his being within that cost. Mr. Raymond.

Whether or not the suggestion here made is wise, may be a matter of opinion; it outlines the procedure the writer would follow.

With this change, and possibly without it, the paper seems to be one of the most valuable that has recently appeared for the study of the young engineer.

F. Lavis, Assoc. M. Am. Soc. C. E.—The speaker thinks that it was unnecessary to offer any apology for the presentation of this paper on account of the fact that the perusal of it might be loss of time to some experienced engineers. An engineer who is interested in railroad location as a science cannot fail to be interested in the development of such a low grade line through a very rough piece of country. The speaker does not feel sure that the low grade was warranted, but will refer to that later. Mr. Lavis.

Mr. Lavis.

The literature on the practical aspects of railroad location is chiefly conspicuous by its absence, and a description of this kind, of actual work performed, is therefore valuable. It would have enhanced the value of the paper considerably had the author been able to go into more details of the work, even at the risk of saying something which might have been known to someone else, and to have shown, if possible, a map of the various preliminary lines run and their relation to the final located line, and to have given some details of the cost of the surveys and the time consumed in making them.

Mr. Taylor states as his fourth principle, in making such surveys as this, the necessity of making "a diligent study of the whole country." Evidently, he did this, before the completion of the work, but fuller details of how much country the reconnaissances covered, how much time was spent on them, and how the final results were deduced from these observations, would have been invaluable, to experienced engineers as well as to students.

It hardly seems possible that railroads to-day are located in any other than a big, broad-minded way, with time and money enough to do the work thoroughly, and to thrash out the country until absolute proof is obtained that the best line has been selected.

The speaker, however, knows from a rather extended experience that, in spite of all that should be known of the increased cost of operation of badly located roads, many miles of railroad are being located and built now in the same old narrow-minded way, and this is the cause of the large expenditures which are being made for relocations and revisions. Many men, investing in railroad projects, still believe that any money spent on engineering, and especially on location surveys, is just so much money wasted.

An instance has come under the speaker's observation recently. On what is now part of a trunk line, and between two points, about 200 miles apart, by the present operated line, a new line has been located, saving more than 30 miles of distance, with ruling grades of one-half of those on the older line, saving nearly 3 000 ft. of rise and fall, and having less than half the total degrees of curvature of the older line. This does not necessarily imply a criticism of the older line, as the speaker does not know any of the conditions under which it was located and built or the relative cost of the two lines, but it illustrates what can be accomplished by a thorough study of the country and by properly conducted surveys.

It seems strange that the author has offered no explanation of the fact that so many changes were necessary after construction was started. It appears that the route had been under consideration, more or less, for the past twenty years, and surely, at least a decision as to ruling grades and the general location of the line, if not of the minor details, should have been reached before construction was commenced.

Mr. Lavis.

It is stated that, when it was found that 0.65% grades could be obtained on this new line, this required changes in minor grades on the part on which construction had already started, and caused the officials of the Louisville and Nashville to determine to make some grade revisions on the old lines which this link connected.

Surely this new line and the whole freight division of which it is a part should have been studied as a whole, to determine the ruling grades possible on the whole division, and a thoroughly comprehensive survey should have been made before construction was started.

One of the grave errors, even to-day, in a great deal of the grade revision, is that it is not comprehensive enough, and the work is done in little pieces.

It does not necessarily follow that, because a comprehensive scheme has been laid out, the work has to be undertaken all at once, but the work done should be governed by its relation to the whole.

It is hardly possible, from the information contained in the paper, to estimate the value of the change from the 1% to the 0.65% grade on this short piece of line, without knowing its relation to the operating division of which it will form a part.

In speaking of the rates of grade, the author uses indiscriminately rates per mile and rates per cent. Rates of grade in this country, at least among engineers, are now spoken of almost exclusively in rates per cent., and it would seem advisable to use this method of nomenclature exclusively, at least in an engineering paper.

In reference to the Missouri prairies: The word, prairie, generally conveys to the average reader an idea of a comparatively level plain, but some of the most difficult location can be found through the Missouri and Iowa prairies, especially for low grade lines. Mountain ridges may be bucked through, but the long rolling country, with differences between ridges and valleys often amounting to more than 200 ft., such as found through Missouri, broken up by cross-drainage, presents some of the most difficult problems in railroad location. The line has to go down into valleys and over ridges, and the slopes are almost always just too steep for the grades one is trying to get.

Mr. Coverdale.

W. H. COVERDALE, ASSOC. M. AM. SOC. C. E.—The author is to be congratulated, both upon his location of the line between Knoxville and Jellico, and upon his description of the work. The locating engineer always delights to narrate his troubles; but, unlike the author, he does not always succeed in making the narration attractive.

The topography of this section of country, as may be seen from a glance at the map, is enough to strike terror into the heart of the engineer who is looking for a railroad of easy grades and tangential alignment between the points named.

Not only does the backbone of the Cumberland Range interpose a rugged barrier some 500 to 600 ft. higher than either terminus, but

Mr. Coverdale.

series of subsidiary ridges, parallel to the axis of the mountain and divided by deeply-eroded valleys, point their forbidding fingers across the path; and precipitous torrents, at widely-varying elevations, roar their challenge of defiance.

The author, in all probability, has secured the best location which the country—and his company—affords. He states frankly that the grade reduction, which left one pusher grade on the road, was an afterthought, considered only when much construction work had been done; hence the relation between the maximum ordinary and pusher grades needs no criticism. All in all, the result of his work can be but a matter of congratulation. The method by which the result was secured is the subject matter of this discussion.

In any topic as comprehensive as that of railroad location, much of interest must necessarily be omitted from a paper, which the author does not intend as an exhaustive treatise; and, indeed, the author states that he has not mentioned that part of the work upon which he spent most of his energy. This probably means the reconnaissance feature, as he emphasizes the importance of a diligent study of the whole country; and if this assumption be correct, an ignorant, but interested, reader of the paper would respectfully ask that, before the discussion be closed, the author give some idea of the amount of such work which was necessary in the development of this line. A thorough reconnaissance and the method of selection of the primary points on the route is not a narrow and detailed question, but a broad determination of policy upon which must rest the vindication of the located line. The fact that the author speaks definitely of the features of Copper Ridge for a distance of 30 miles from the point where he crossed it throws a little light on the scope of such work.

About seven years ago the speaker drove more than 20 000 miles, looking for a 20-ft.-per-mile grade for the Pittsburg, Ft. Wayne and Chicago Railway, between the Ohio River and the prairie country, some 200 miles to the westward, and he recalls the narrow and prejudiced state of mind in which he was when he began the work. A summit, lower than the one over which this line was laid, and in plain sight of it, meant nothing to him but that the locating engineer had been in error, while a detour of 10 or 50 miles, if it afforded the required grade, was the one route which should have been considered seriously. Such matters as excessive cost of construction, failure to serve important centers of population, greatly increased mileage, etc., were mere bagatelles when weighed against the reduction of ton-mile cost.

However, a little good advice from his chief engineer soon put him in a more tolerant attitude, and he began to realize that topography is but one of several factors, and too often an unknown one at that, in the transportation equation.

Mr. Coverdale.

This suggests a second point to which the speaker wishes to call attention, namely: Men of a previous generation must not be judged by present standards, and this is equally true of technical skill and of morality. Who shall say that the old Portage Road across the Allegheny Mountains was not the proper answer to the stage-coach problem? Who shall say that Hernando Cortez and Nero were cruel men?

The author compares his grade of 34 ft. per mile with that of 66 ft. per mile existing on rival roads. He also states that his total average rise and fall per mile is only 21.5 ft., as against 59.2 ft. for the old road. He states, further, that his road is smoother than an unnamed prairie road by 42 per cent. These facts are pertinent and interesting, but they are disjointed, or, rather, isolated; and, in all fairness, the inference that the first railroad through this country should have been constructed on the location described by the author must be avoided.

Many other facts must be added to those stated by the author before the chain of cause and effect can be completed, as, for instance, the cost per mile which secured the 34-ft. grade, as against the cost of the 66-ft. grade, the length of the respective routes which exceeded the straight-line distance by 34 and 55% in favor of the old route, and the general traffic and commercial conditions obtaining at the respective dates of construction.

Because men built railroads on a 1% maximum grade fifty or twenty years ago, it by no means follows that they built badly, and their work must not be judged by the conditions under which work is done to-day. Increased traffic, competition, reduction in ton-mile revenue and other considerations to-day make easy gradients imperative and economic, which, a generation ago, would have been the height of folly; and it may even be possible that this "widow's mite" railroad, running up and down and across country, over the hills and hollows—on which, it may be, the old rails have a hard time to keep the fire-box off the ties—may have represented as nice an adjustment of means, methods and results as the author's modern road.

In regard to grade revision and the method of determining the amount which could be spent economically for tunneling, it is noted that the author's estimates are based upon a 2 170-ft. tunnel at Copper Ridge and a 700-ft. tunnel at Black Oak Ridge No. 2. The actual construction work, however, was not carried out on these lines. The town of Clinton, with its contiguous 700-ft. tunnel, was weighed in the balance against the town of Dosset and the claim of the Oliver Springs Branch, and found wanting, and so a sweeping change of location was made and a tunnel 3 520 ft. long was substituted for the 700-ft. tunnel.

The total length of main-line tunnel, therefore, is 5 690 ft. instead

Mr. Coverdale.

of 2 870 ft., or practically double the length used in estimating. It is beside the question to offset the additional cost of main-line construction against the saving on the branch line because main-line construction costs must be justified by main-line traffic results.

Furthermore, although by this plan the grades on the branch line were reduced to the main-line ruling grade against south-bound traffic, and although, theoretically, this is very nice, yet grades of coal branch lines are by no means as important as main-line grades, and for several reasons:

1.—Through main-line freight trains do not travel over such branches, nor stop at such junction points to take on loads or set off empties, except in rare instances.

2.—Trains running light and filling to capacity at local points are rarely a factor in grade determination.

3.—Local trains deliver empties to mines and collect loads therefrom, and seldom work up to full engine capacity.

4.—Coal branch grades are with, rather than against, their traffic, and a grade of 34 ft. per mile, over which to shove merely empty gondolas is an expensive luxury.

It appears, therefore, that the author's estimate of what the 34-ft. grade would cost should have included all main-track work as actually built. Since it is not apparent from the paper what the increased cost of the 5 690 ft. of tunnel is over that portion of the original line which it displaces, it would not be pertinent to subject the author's estimate to detailed examination.

It is pertinent, however, to call attention to the idea which pervades the argument expressed on page 481: "Operating expenses vary directly as the train-mileage," and to be satisfied that this premise be a sound one before accepting the conclusion.

The *Transactions* of this Society contains a memorial of that great railroad economist, Albert Fink;* and even from those brief notes may be learned the fact that his first analyses of railroad operations were embodied in the annual reports of the Louisville and Nashville Railroad about thirty years ago.

Mr. Fink's writings upon this subject are still classics, and it is of interest to note that the officers of the very railroad, the records of which contain so much that is practical and scientific, acquiesce so readily in the empirical theory which the author states as a fact.

Revenue train-mileage is a unit of service by which may be measured the cost price of transportation. To say, however, that this cost price varies directly as the total amount of transportation sold, is to disregard all the primary-construction cost of the property, much of the maintenance, and a considerable amount of the costs incidental to train service and general expenses.

* *Transactions*, Am. Soc. C. E., Vol. XLI, p. 626.

Mr. Coverdale.

The Interstate Commerce Commission classifies railroad operating expenses under four main headings:

1st.—Maintenance of way and structures,
2d.—Maintenance of equipment,
3d.—Conducting transportation,
4th.—General expenses.

A brief examination of these expense items will show to what extent they vary as train-mileage:

1.—"Maintenance of way and structures expense" "fluctuates, not with the rate of wear, but with the rate of renewal," says Mr. Eaton, the statistician of the Lehigh Valley Railroad, in his book on "Railroad Operations." The character and amount of traffic determines in a general way the types of track and bridge structures, but large sums are spent annually for maintenance of such structures, and appear in the figures which the author has used, which have nothing whatever to do with train-mileage.

Widening banks, sodding slopes, cutting weeds, repairing fences and track signs, many tie renewals, painting bridges and buildings, renewing bridges and buildings destroyed by fire, flood, or lawlessness, the substitution of steel and masonry for wooden structures, grade reductions, revisions of alignment and other betterments of magnitude not chargeable to capital account: All these items of operating expense depend in no way upon the train-mileage.

The "maintenance of way and structures" item, for the Louisville and Nashville Railroad, amounted, in 1902, to about 22% of the entire charge against operation; and, of this amount, only $33\frac{1}{3}\%$, or 7% of the whole, is chargeable to train-mileage. This apportionment is made on the basis of the Pennsylvania Railroad methods, and includes all renewals of rails, frogs and switches, $33\frac{1}{3}\%$ of cross-tie renewals and 10% of repairs of roadway, track and bridges. All repairs of buildings, docks and structures, other than above, are taken as independent of train-mileage.

2.—"Maintenance of equipment expense" varies with engine-mileage, freight and passenger car-mileage, but not with train-mileage. It depends upon alignment and grade of line, weather, speed and accidents; it includes cost of marine-equipment maintenance, interest on shop investments, etc. This item, for the Louisville and Nashville Railroad, amounted, in 1902, also to about 22% of the entire charge against operation, and cannot be said to vary directly with train-mileage. It is noted that, while repairs to locomotives, passenger and freight cars, etc., are the direct result of operation, yet a unit of maintenance, much more exact than train-mileage, is now in general use.

3.—"Conducting transportation" is usually subdivided into twenty-five or more accounts, including fuel, water, locomotive and

Mr. Coverdale.

miscellaneous supplies, wages of engineers, firemen and roundhousemen, switchmen, flagmen, station supplies, advertising, telegraphing, car-mileage, injuries to persons and property, etc., etc., and only one of these accounts, namely, train service, which covers the wages of conductors, baggagemen, brakemen and flagmen, can be said to vary directly with train-mileage. This item, for the Louisville and Nashville Railroad, amounted, in 1902, to about 52% of the total operating expenses, and of this amount only 13% varies with train-mileage.

4.—The item "general expenses," of course, does not vary with train-mileage, so that, in all, about 14% of operating expenses depends upon train-miles run, and upon this slender thread hangs all the weight of the author's argument.

Several other points are suggested, but they are of a more or less detailed nature, and are, in consequence, omitted. The speaker has derived both pleasure and profit from a perusal of the paper.

Mr. Linton.

HARVEY LINTON, M. AM. SOC. C. E. (by letter).—The writer finds little to add to Mr. Low's excellent description of the location of the Clinch Valley Division of the Norfolk and Western Railroad.

Whatever reconnaissance was made in 1886 afterward had to be modified considerably. The line between St. Paul and Little Tom Tunnel was fixed upon before any survey, other than the writer's reconnaissance, had been made.

Early in the spring of 1887, the writer rode from Graham, Va., to Big Stone Gap, Ky., after having spent about two weeks in charge of the first surveying party in the field, on Bluestone, above Graham. Arriving at Big Stone Gap, he met there the late R. L. Cobb, M. Am. Soc. C. E., then Chief Engineer of the Louisville and Nashville Railroad. After a few days at Big Stone Gap, the writer rode back, over the line, looking into the valley of Guest River, and exploring the ridge between Russell and Bull Creeks. A good aneroid, and a pedometer which had been calibrated with the horse's step to read miles, were of great service in this work.

It was soon evident that something unusual would have to be done, to get a railroad location from Clinch River to the valley of Powell River. Guest River, from its mouth up, although described previously* as "the natural gateway to this region," was simply impossible. The valley of Bull Creek is like a deep cañon. A location down the Clinch River and up Bull Creek did not look promising. After starting with an ascending grade from Lick Creek (St. Paul), Creagan Tunnel is necessary, to get from the valley of Whetstone Creek to the valley of Russell Creek. After that, the "awful" profile that the location developed had to be accepted; there was no alternative. This the writer stated to Mr. Coe, on his return to Roanoke, when he made

*In d'Invillier's Geological Report to the Norfolk and Western Railroad Company.

a large sketch, in red and blue pencil, of the location which was afterward adopted. Mr. Coe accompanied the writer on his next trip, and was convinced that this was the best location. Later surveys proved it. Altogether, it was an interesting experience. Mr. Linton.

Mr. Oramel Barrett in 1887 made a survey down Guest River, so as to hold that route against the Norfolk and Western Railroad Company. The writer made no survey down Guest River, below the present line of the Norfolk and Western. The Lewis Creek and Thompson Creek lines were developed later in the season, after surveys had been in progress some time.

The summit of Bluestone (near the highway) was found, by aneroid, to be, approximately, 100 ft. higher than the summit of Wright Valley Creek. This led to the adoption of the location over Wright Valley Creek Summit.

Mr. Low's remarks relative to the organization and methods of field parties are very good. Before the surveys for the Clinch Valley location were finished, notes were sent to the office, at Tazewell Court House, and there plotted.

W. T. Forsythe, Esq. (by letter).—In the location of the Knoxville, La Follette and Jellico Railroad the writer does not think the difficulties to be overcome were as great as those presented in the Clinch Valley Division of the Norfolk and Western Railroad. The topography of the country, or, rather, of the valleys, is very dissimilar, and is generally supposed to be caused by faults in the vicinity of Pineville, Tenn. The Middlesboro Basin is practically the same as the basin at Big Stone Gap. This is separated from the Jellico Basin by an unproductive belt, about 6 miles in width, where the Lee Conglomerate is found as the overlying rock, under which, as under all conglomerate rocks in Pennsylvania, no bituminous coal is found in place. Jellico is on or near the eastern edge of the Jellico Basin, and the coal measures in that basin are the upper measures of the Pocahontas Series, while those of the Middlesboro Basin represent the No. 3 or Pocahontas vein. The upper measures are not nearly as pure as the lower measures. Mr. Forsythe.

The coal-bearing rock series of this formation thicken on the southeast border, the elevation of the veins is increased, and they finally vanish.

The rocks east of a certain point have been eroded more, being softer, until the limestone formation, peculiar to the Clinch Valley, is reached.

The general direction of the coal measures, when no displacement has occurred, varies from N. 30° E. to N. 40° E. in this section, which is the general course until the Susquehanna River in Pennsylvania is reached, when it is deflected more to the east, and the anthracite coal series prevail, no anthracite coal being found west of the Susquehanna or east of the Lehigh.

Mr. Forsythe. The Clinch Valley Railroad, being located southeast of the coal-bearing series of rocks, is in the district most affected by erosions. Thence the streams are more winding. The upheaval of the limestone caused a diversion at Young's Summit, and all along the line in Tazewell County as far as Cedar Bluff this rock asserts itself. The particular feature which the writer wishes to show is that the strata southeast of this line of the Pocahontas vein are broken, from Birmingham, Ala., to New York State.

Northwest of this line the strata are nearly horizontal, and are broken very little for a distance of nearly 100 miles. The coal beds thin out toward the northwest and dip in that direction, with a few anticlinals, until they are too deep to be workable. The coal-bearing rocks of the Kanawha Series become the producing strata, until they take the same dip in a northwest direction, and are overtopped by the Pittsburg formation, which is the upper formation and is distinguished from the other by being semi-anthracite in character, in many places.

The rocks through which the Big River forces its way are not displaced, as in the Clinch River, and therefore the sinuosities are not as common.

On the Clinch Valley Division of the Norfolk and Western the Creagan Tunnel is the last of the limestone ridges penetrated. The time allotted to the survey of the territory lying between the mouth of Thompson Creek and the mouth of Guest River, and up Guest River to the vicinity of Coeburn or the mouth of Big Tom, was scarcely sufficient.

A careful survey should have been made from Keyser Tunnel, near the mouth of Dump Creek, over the dividing ridge between Dump and Lick Creeks, and again over the ridge to the headwaters of Little Tom. Thus, the tunnels, when necessary, would have been through sandstone altogether, and the coal-bearing strata would have been reached in shorter distance from Graham, and the coal fields traversed a longer distance. The fall of Guest River, between a point below the mouth of Big Tom and its junction with Clinch River, is at the rate of 120 ft. and more per mile.

Russell Creek Valley is 100 ft. higher than Bull Run Valley at opposing points. Therefore, it might have been better to have made one tunnel from Whetstone River into the Russell Creek Valley and another tunnel into the Bull Run Valley at the head of Lawson's Hollow, where the 150-ft. trestle was built, being the nearest of the high trestles to the Little Tom Tunnel; thus, instead of the Creagan Tunnel, the heavy cut in the gap into Russell Creek Valley, the Big Bull and Little Bull and Holbrook Tunnels, the dip in the crossing of Russell's Creek, the high trestles (two of 125 ft. and two of 150 ft. each) might, perhaps, have been avoided, and two tunnels, rather

longer than any one of these, but not measuring more than 75% of the total tunnel distance, would have been substituted. Mr. Forsythe.

The saving in cost might have been considerable, and the gradient reduced to at least 1.25 per cent. As mentioned before, the time was short and the season very unfavorable for a survey, as there were many rains during the twelve weeks devoted to this region.

The writer has made extended examinations of the eastern outcrop, from a point near Chattanooga, Tenn., to Broad Top, Pa., and has been confirmed in the developments when found. The Cumberland Plateau is mostly an unbroken mass of rock, and the coal measures have few faults and very little parting, or bony coal; both to the northeast and southwest some displacements are found, and in such places more bone is found and sometimes the veins have two partings instead of one. Nothing would have been gained by going down to the mouth of Bull Run—there would only have been more grade to overcome. It rises most abruptly in the upper portions of the valley, and the fall is very gradual from Lawson's Trestle down.

W. D. Taylor, M. Am. Soc. C. E. (by letter).—The writer congratulates himself that, with possibly a single exception, this paper has met with so fair and frank a discussion. Mr. Taylor.

Many of the points brought forward are suggestive, and the writer regrets that there could not have been time and opportunity for a discussion of this nature during the lay-out of the work.

Engineers connected with railway work have been slower, perhaps, than others of the profession in abandoning, toward their own work, the old unscientific attitude, which maintains that "it's right because I fixed it so." A prominent railway president, of a western road, who is not an engineer, insisted, not a great while ago, in conversation with the writer, that even now they were making small progress in this direction.

Though no engineer can project a work of this magnitude, which involves as many conflicting elements as extensive railway location in difficult country always does, and which is laid out in the hurry that was unavoidable in this case, and feel absolutely sure that serious errors have not been made, yet, the writer trusts that no really essential element was overlooked in this lay-out.

Before discussing the questions raised by the writer's critics, a few points not included in the paper will be mentioned.

A letter of instructions, as to their duties, sent to all resident engineers on construction on this line, may be found in *Engineering News*.*

Clauses in the specifications for construction which vary somewhat from the usual forms were as follows:

"Stumps shall be grubbed for 2 ft. back from the slopes of cuts, from ditches, and from new channels for waterways.

* March 5th, 1903, page 217.

Mr. Taylor.

"Clearing shall be paid for separately. Grubbing shall be paid for in the price paid for excavation.

"The price paid for clearing is understood to apply to the heaviest clearing to be encountered along the line. When the clearing is lighter, it is to be rated by the engineer, so that at a point where only half the work done on the heaviest sections is necessary it shall be rated accordingly.

"Embankments shall not be built with wheel-barrows, grading machines, or by casting, or by any process which does not thoroughly pack the material. Dump-cars may be used only on very high fills and upon the special instructions of the engineer.

"All embankments shall be kept to the full width of cross-section at all heights while being built, with the sides at least as high as the center of the roadbed.

"In constructing any embankment, except one built of hard rock material, no back dumping from carts is to be allowed, but the embankment is to be built either in 6-in. layers or less, or to be so sloped that the teams will constantly pass over all material as soon as it is dumped."

These restrictions as to the manner of building embankments did not seem to have a very ill effect on the construction prices, as will be apparent from an examination of the contract prices given in Table No. 5. The excavation was all paid for as measured in excavation, with a free average haul of 500 ft. The overhaul clause reads as follows:

"Average haul shall be computed for each cut separately, and short haul from one cut shall not offset long haul from another cut. The average haul from any cut shall be the distance between the center of gravity of that cut and the center of gravity of the fill which the material from that cut makes."

The two branches and the work on the main line north of La Follette were let to one firm of responsible contractors; the work from La Follette to Beaver Creek to another; and the work from Beaver Creek to Knoxville, and including the Knoxville yards, was let to another. The price of ordinary labor, most of which was done by negroes, varied from $1.00 to $1.35 per day.

The prices paid in the contracts varied between the limits given in Table No. 5.

Timber and stone were fairly well distributed on all the lines north of La Follette. Suitable timber was, for the most part, white oak, which was too small for trestle stringers. South of La Follette all timber, except that of a very temporary nature, had to be imported, and stone for masonry was not very well distributed. For example, the stone for the masonry of the Clinch River Bridge had to be brought in barges from near Clinton. No transportation for either freight or passengers was allowed on the Louisville and Nashville. Portland cement of the better brands was worth $2.85 per barrel in Knoxville.

Three civil engineer students, of the University of Wisconsin, graduating in 1904, who were employed as transitmen and topographers

TABLE NO. 5.—CONSTRUCTION PRICES ON THE KNOXVILLE, LA FOLLETTE AND JELLICO RAILROAD.

Clearing, per acre	$18.00	$20.00
Earth excavation, per cubic yard	0.17	0.19
Loose rock excavation, per cubic yard	0.35	0.37
Solid rock excavation, per cubic yard	0.67	0.70
Chert excavation, per cubic yard	0.28	0.28
Water excavation, per cubic yard	0.75	1.50
Tunnel excavation, per cubic yard	2.50	3.00
Iron, in trestles and foundations, per pound	0.04	0.04
First-class, stone, bridge masonry, per cubic yard	8.00	10.50
First-class, stone, arch masonry, per cubic yard	10.00	11.00
Stone, culvert masonry in natural cement	4.50	5.50
Stone, culvert masonry, laid dry, per cubic yard	3.25	4.50
Rip-rap, per cubic yard	1.00	1.50
Stone paving (in culverts), per cubic yard	1.50	2.00
Concrete in foundations, Louisville cement, per cubic yard	4.25	6.50
Concrete in piers and culverts, Portland cement, per cubic yard	6.75	7.50
Concrete in arches and walls, Portland cement, per cubic yard	6.75	7.50
Temporary trestles, per 1 000 ft., B. M., including iron	15.00	17.00
Concrete packing in tunnels, natural cement, per cubic yard		6.00
Dry packing in tunnels, per cubic yard	1.50	2.25
Piles in foundations, per linear foot	0.40	0.50
Timber in foundations, per 1 000 ft., B. M	24.00	27.50
Piles in pile bridges, left in work, per linear foot	0.40	0.40
Pile trestles exclusive of piles and stringers, per 1 000 ft., B. M	28.00	28.00
Framed trestles, per 1 000 ft., B. M	28.00	28.00
Timber in tunnels, left in the work, per 1 000 ft., B. M	27.50	28.00
Hauling drain pipe, cement, or other construction material especially arranged for, per mile per ton,	0.35	0.44
Overhaul, per cubic yard	0.01	0.015
Necessary haul on masonry stone, per cubic yard per mile, after first mile	0.75	0.75

on this location and as assistant resident engineers on the construction, chose, as the subject of their joint graduating thesis, the location and construction of this road. From the description, in this thesis, of the

Mr. Taylor. geology of the district, by Mr. M. W. Torkelson, are taken the following abstracts, which aptly describe parts of the country between La Follette and Knoxville:

"Big Creek is a peculiar stream. The main body of the stream is made up of three branches, which unite just above the gap. The most important of these flow in exactly opposite directions, the one southwest and the other northeast parallel to the direction of Cumberland Mountain, which lies between them and the valley to the southeast. The third and least important of these branches enters from the northwest flowing nearly at right angles to the direction of the mountain. After cutting through Cumberland Mountain, Big Creek flows through the Silurian and Ordovician rocks of the valley district to the point where its waters are discharged into the Clinch River.

"Immediately to the southwest of Cumberland Mountain is a long and comparatively narrow valley known as Powell's Valley, which extends along the southeast foot of the mountain from its southern end far into the State of Virginia. This valley lies partly in the Chickamauga limestone formation and partly in the Knox dolomite. Between Powell's Valley and the Clinch River to the southeast lie a number of irregular hills, whose greatest elevation is about 1 600 or 1 700 ft. above the sea level. The streams draining Powell's Valley do not flow along the valley, but directly across it, and, cutting through the hills, reach the Clinch River. Probably, at one time, a stream flowed along the valley, but the streams tributary to the main valley gradually cut their way back through the hills, and finally managed to divert the drainage from its old course along the valley, to a new course straight across it. These creeks all flow through very narrow and deep valleys, which are also very crooked.

"After leaving Big Creek Gap, the course of the location is in a southwesterly direction along the valley, partly in the Chickamauga limestone formation and partly in the Knox dolomite, the location being very near the dividing line between these formations. Through this vicinity, between Big Creek Gap and the Village of Jacksboro, there are no abrupt steep slopes, only gentle slopes being encountered along the valley. A short distance below Jacksboro, the location turns to the left until its course is but little west of south, and enters the hilly country. Here the formation is all the Knox dolomite. The hills are rough and irregular, with rounded tops and rather steep sides.

"The Knox dolomite already referred to is the most important of all the valley formations, and receives its name from Knoxville, Tenn., which is located on one of its areas. It consists of a series of blue, gray, and whitish, limestones, about 2 500 ft. thick. Many of these are banded with thin, brown, sandy streaks. Nodules, or masses of chert, are frequent. The decay of the calcareous and magnesian compounds contained within the rocks, leaves a rich, red soil. The chert nodules, being less soluble than the carbonates, remain within the soil, and in places where there has been much decomposition, the cherts are apt to form a large percentage of the soil. The areas of the Knox dolomite are of great extent, stretching in broad belts across the country in a northeasterly and southwesterly direction, generally forming ridges. The age of the formation is great. Some fossils are found indicating that the first depositions, which go to make up the Knox dolomite, occurred in the middle Cambrian time, but the greater part of the formation is unquestionably of Ordovician origin.

"The territory between Walden's Ridge and Knoxville is occupied

Mr. Taylor.

by a succession of parallel ridges and valleys, which extend across the country in a direction about 50° east of north, and all lie coincident with the outcrops of the various geological formations. The ridges are seven in number; namely, Black Oak Ridge, Pine Ridge, Chestnut Ridge, Bull Run Ridge, Copper Ridge, Beaver Ridge and Black Oak Ridge. Of the two ridges called Black Oak Ridge, the one near Knoxville was distinguished in the surveys as No. 1, and the other as No. 2. Throughout this territory the Knox dolomite is exposed in belts, from one to two miles in width, which alternate with other similar belts of about equal width, composed of narrow strips of the Conasauga shale, the Rome formation and the Rome sandstone-lentil. The ridges previously enumerated lie alternately within areas of the older formations, and show very plainly how topography is modified by geological agencies. The two Black Oak Ridges, Chestnut Ridge and Copper Ridge are composed of dolomite; they are very similar to each other and different from the other ridges, which are composed of the older formations. The dolomite ridges are more prominent, broader, and with steeper slopes. They are very continuous, and they all extend for many miles without being cut by stream gaps. Along the lower slopes of these ridges, large springs of clear, cold water are common. These springs are very characteristic of the dolomite ridges. In the valleys, where the ridges on opposite sides are of different formations, the springs are invariably on the side where the dolomite is found. The ridges of the older formations, which alternate with the dolomite, are Pine Ridge, Bull Run Ridge, and Beaver Ridge. The older formations are much softer than the dolomite, and consequently more easily eroded. These ridges are narrow and frequently cut by stream gaps, generally at intervals of two or three miles. The large springs, so frequent in the dolomite, are entirely absent.

"To reach Knoxville from the valley of Brushy Fork of Poplar Creek on the southwest side of Walden's Ridge, it was necessary to cross each of the intervening ridges. The first one encountered was Black Oak Ridge No. 2. This is a ridge having a maximum elevation of from 1 200 to 1 300 ft. above sea level, which, though of no great elevation, is unbroken for many miles.

"Between Lee's Ford and the mouth of Bull Run, the (Clinch) river cuts through three of the parallel ridges, namely, Pine Ridge, Chestnut Ridge and Bull Run Ridge. This made it possible to avoid a crossing of these three ridges, and to touch them only where they slope down toward the river's edge. The manner in which the different formations influence topography is plainly illustrated here. Pine Ridge and Bull Run Ridge, which lie in the more ancient and softer shale formations, have gentle slopes and are cut by frequent gaps. Chestnut Ridge is continuous from Clinch River northeast for more than twenty miles. It is also more prominent and has steeper sides than the other two ridges. Where the location comes in contact with these ridges, they do not exhibit any marked differences in topography, since the location only skirts the lower portion of the slopes remaining where they have been cut by the river, but, even here, Chestnut Ridge has a considerably steeper slope down to the river, and the resistant qualities of the dolomite are shown by rapids in the river.

"At the mouth of Bull Run, the Clinch River makes a very abrupt turn to the right, through an angle of more than 90 degrees. At this point the stream strikes squarely against Copper Ridge, the most important range of hills between Walden's Ridge and the City of

Mr. Taylor. Knoxville. The greatest elevation of this ridge is not more than from 1 200 to 1 300 ft., but the ridge is very steep on its northwest side, with a much more gentle slope opposite. At the point where the location crosses the ridge are two large sink-holes, also a number of smaller ones, probably caused by the removal of the soluble constituents of the dolomite by solution in the ground-water, and a subsequent sinking of the surface soil to fill the cavity. Near the gap, on the east side is a large cave of unknown extent, and it is probable that the falling in which caused the sink-holes went to fill similar cavities.

"Between Beaver Ridge and Black Oak Ridge lies Hines's Valley, entirely within the dolomite formation. This valley has a general slope toward the southwest, but no stream drains its entire length, the drainage being to the northwest, through Beaver Ridge into Beaver Creek, and forming a very interesting instance of what is known in geology as stream capture. The elevation of Beaver Valley is about 60 ft. less than Hines's Valley. Originally, Hines's Valley was drained by a stream flowing its entire length to the southwest. But tributaries of Beaver Creek cut their way further and further into the soft rocks of Beaver Ridge, finally cutting entirely through the ridge and causing streams which had formerly carried their waters southeast into the Hines's Valley stream to flow northwest into Beaver Creek. These streams, which flowed through the soft shales, cut their channels faster than the main stream of Hines's Valley, which lay in the harder dolomite, and eventually caused the main stream to leave its old bed and flow northwest through the gaps in the ridge. Since then the slopes in the valley have been changed. The prevailing slope is still downward toward the southwest, but made up of a succession of shorter slopes which pitch downward alternately to the southwest and the northeast.

"All the creeks draining valleys in the dolomite formation are similar. In the lower parts of their courses, the last mile or two, the streams are much more rapid than higher up. In the upper part they flow gently, with many bends, through meadows of alluvial deposit, but, lower down, the fall is more rapid, and their courses more direct, over exposed dolomite ledges, to the main stream."

There have appeared in the discussion so many inferences which were contrary to fact, even though they were seemingly warranted by some of the writer's awkward expressions, to one of which Mr. Beard calls attention, that it would seem to be well for anyone who wishes to make a special study of this location, and whose impressions as to the facts are not cleared by the closure, to inspect the topographic sheets of the Geological Survey covering the area traversed by the road and its branches. They are the Williamsburg, Briceville, Loudon, Knoxville, Cumberland Gap and Maynardsville Sheets. In order to give a more adequate idea of the character of the country and of the work, a few photographs, Plates XXII, XXIII and XXIV, have been inserted.

The discussion by Mr. Low in regard to the Clinch River Division of the Norfolk and Western is particularly interesting to the writer. This discussion, taken in connection with his discussion of the paper

ROCKHEAD IN BIG CREEK GAP THROUGH CUMBERLAND MOUNTAIN.

"Heavy Railway Construction in Wyoming,"* seems to warrant the assertion that it is not possible to escape expensive work in this territory, even when using 90-ft. grades in both directions, temporary structures, and when running generally with the drainage. No doubt the topography at and near the headwaters of the Clinch is somewhat rougher than along the Knoxville, La Follette and Jellico, but the latter country is of the same general character as the former, and the Knoxville, La Follette and Jellico was built with permanent structures, with very much lighter gradients, and generally across drainage. The line of the Knoxville, La Follette and Jellico lies very low, relatively to the adjacent country. A gravity-supply water station, ample throughout the year, was easily available right on the summit of the Clinch-Cumberland Divide, the highest point on the line. Mr. Taylor.

The writer does not know what the cost per mile of the Knoxville and Ohio Branch of the Southern would be, in existing condition, and, judging from his experience in railway appraisals, is quite sure that no one else does. But the line has the same number of tunnels as the Knoxville, La Follette and Jellico, two of which are quite long, and has a quantity of quite heavy construction. The line probably could not be reproduced with properties new and with permanent structures at a less cost per mile than $40 000, exclusive of terminals and equipment.

It is no doubt due to another of his awkward expressions that the writer's critics have gotten a somewhat exaggerated idea of the expensiveness of the line. The cost data for the main line, given on page 489, includes the cost of expensive terminal lands, yards, and station buildings within the City of Knoxville. The detailed figures are not at hand, but probably between 20 and 25% of the total amount will be expended within the limits of that city.

Further, by far the costliest part of this line, barring the construction of the tunnels, which have received so much attention, was at the head of the Narrows and along Hickory Creek, just south of the Narrows, where there was only one route possible, and where there was no question whatever concerning gradients.

The writer would gladly accept Mr. Walter Watson's suggested change and interesting addition to the rules outlined for location work, also Professor Raymond's suggested change in verbiage.

The writer is quite sure Mr. Coverdale did not catch the import of all the reasons given for the change of the location from the route *via* Clinton to the route *via* Dosset. The main line, itself, was shortened nearly ½ mile by that change, and the alignment very much improved, in addition to the other advantages explained.

Not one cent was expended to secure 34-ft. grades on the branch lines over which to shove empty gondolas.

* *Transactions*, Am. Soc. C. E., Vol. XLVI, p. 17.

Mr. Taylor. There is no doubt, as some of the writer's critics have suggested, that parts of the road were put under construction too soon, before a definite detailed policy of development had been decided upon. There were such urgent reasons for haste in some of these matters that grading forces were put on parts of the line before all the surveys were connected, the reasons for which can be easily divined by the initiated.

Since the writer has seen so much adverse grade used, which could have been excluded at no expense at all, but, by the exercise of a reasonable amount of care, he confesses to personal antipathy to such grades. Although all that Mr. Watson says, in regard to using undulating grades with sags dropping from 30 to 40 ft. below a generally level gradient, is true enough, the writer suggests that the evil effects of an adverse grade can hardly be over-estimated in the lay-out of a road for slow and heavy traffic on long ascents where the grade rate is the maximum or approaching the maximum; that is, where the gradient exceeds, for the slowest and heaviest trains, Class *A*, of Wellington's classification. The evil effect of such an adverse grade consists, for the most part, in that, wherever it is possible, the total elevation could be surmounted with a uniform gradient of less rate.

The grade profile shows that in comparatively level country, where the vertical elevations surmounted were not so great but that momentum could be used effectively in surmounting hills, adverse grades were permitted in order to reduce the cost of the work. But on long ascents on the steeper gradients momentum grades have no place, and, in such cases, adverse grades were excluded.

One of the writer's critics contends that, as the natural drainage route from La Follette to Knoxville led down Big Creek and the Clinch to Clinton, that route should have been followed. One of the usual consequences of leaving a natural drainage route in railway location is adverse grade, and the amount necessitated on the original line between La Follette and Clinton was cited by the writer to show the extent of the penalty paid, which amount, though large enough at one point to be both objectionable and expensive, seemed to be warranted by the advantages secured. Besides, as Mr. Torkelson shows, in the quotation from his paper, the line followed a natural drainage route for the first few miles south of La Follette, while the stream, Big Creek, followed a route which in one sense might be called artificial. To the writer, the reasons given for the selection of the route between La Follette and Clinton, on the original location through Clinton, seem conclusive enough. The location that Mr. Low describes also left the natural drainage route. And the penalty paid in this case was, not only adverse grade, but the introduction of a grade of 90 ft. to the mile, and it was the only one on 100 miles of road exceeding 58 ft. to the mile, opposing the direction of probable

FIG 1.—BEGINNING CONSTRUCTION, JONES BLUFF ON COPPER RIDGE.

FIG. 2.—MASONRY AT HICKORY CREEK CROSSING NO. 3; AND NORTH PORTAL OF TUNNEL NO. 2.

heavier traffic. This is certainly very objectionable, but, without opportunity to make a careful investigation, the writer would be slow to say that the location made was ill advised, for conditions, and not merely theories, are to be dealt with in railway location everywhere, and especially in country of the Clinch River variety. If the river line was taken, the locating engineer on Mr. Low's road was probably confined, on Clinch River between Sword's Creek and Cleveland, as the writer was on Big Creek below La Follette, to a choice between a crooked and expensive line, unduly lengthened from following the windings of a tortuous mountain stream, and a more direct line, constructed for the most part in tunnels and on bridges. Mr. Taylor.

The primary object in the location of this line was, not to reach Knoxville, though that was of great importance, but to effect through connection with the Atlanta, Knoxville and Northern to the South. The road enters Knoxville on the northwest side of the city, passes through the western edge of it, and effects the junction with the Atlanta, Knoxville and Northern at the southwest edge of the city, where large freight yards are provided for, and then runs northeast and north 1¼ miles to the center of the city, where are the city freight and passenger yards and stations. Thus, through freight will pass only through the western edge of the city. If the through route had been established *via* the line now occupied by the Southern it would have been, on this account, 8, and not 11, miles shorter. If the route had been laid out on 1% compensated grades, *via* the general route now held by the Southern, retaining the present crossings of valleys and summits, and putting sufficient development into the line to gain the necessary elevations, that route would have been lengthened enough, between Clinton and Knoxville alone, to account for all but 1½ miles of the present difference in length. The cost of laying out a road through the district, on the general route occupied by the Southern, on compensated grades of 1% south-bound and 1.15% north-bound, would not be warranted by any traffic that any road in the district can reasonably expect for years to come.

Further, the Atlanta, Knoxville and Northern runs southwest out of Knoxville, while this road, from the point, *P*, in the open valley of Beaver Creek, runs in a general direction due east 14 miles to the Atlanta, Knoxville and Northern connection at Knoxville. From this point, *P*, to a point immediately south, 12.5 miles distant on an air line, and 19 miles below Knoxville on the Atlanta, Knoxville and Northern there is an excellent route available, under the same restrictions as to gradients, but which route involves the construction of a tunnel under Black Oak Ridge and a bridge over the Tennessee River. This cut-off (say, 15 miles in actual length) would shorten the through route at least 17 miles, and there was little doubt in the writer's mind that the traffic of the road at no very distant day would

Mr. Taylor. warrant its construction. The probability of the eventual building of this cut-off had its due share in the decision on the present route south of Coal Creek.

By some heroic construction, between 2 and 3 miles of distance could be cut out by the plan, very pertinently suggested by Mr. Walter Watson, of cutting out the development at Copper Ridge. That the necessary tunnel to carry out his suggestion would be very much longer than he estimates is due to the fact that the part of the map on Plate XXI, at the extreme up-stream end of Beaver Creek as shown, is a mere sketch, and is inaccurate. The north side of Copper Ridge is precipitous and badly broken, particularly at the elevation at which such a line would strike it, and, though there is some, there is a dearth of, supporting ground up the Clinch toward Lee's Ford, as may be seen from Plate XXI. When the traffic of the road warrants the building of the cut-off mentioned above, it may be that this revision may also be warranted.

The element producing the greatest doubt in the writer's mind as to the advisability of reducing the south-bound grades has not been mentioned by any of his critics. If the original location on the 1% grades should have been retained, then, in the writer's opinion, it should have been done on account of the commercial and operating value of an early opening of the road, rather than because the immediate expenditure to effect the reduction was not justified by the results to be gained. Certainly, the value to the system, of this link which should unite its separated parts, for the period through which the opening of the road was necessarily postponed by this reduction, was very much more than the bare interest on the capital already expended in construction.

The writer wishes to say that the general officers of this road were in no wise misled. The President of the Louisville and Nashville Railroad seems not to believe that he needs an engineer to advise him as to the operating value of a given grade reduction on any part of his system, and, in the writer's opinion, that belief is wonderfully well founded. He never saw the figures which give alarm to two of the writer's critics, but when he saw the figures as to what the reduction would cost the writer still thinks he fully understood. Every important step in the lay-out of the road was fully discussed with the Chief Engineer of the Louisville and Nashville, and with other engineers who knew their business. Even if custom on railways permitted, it is doubtful if any one man ought to undertake to decide important steps in expensive and extensive railway construction, without full and free discussion with several traffic and construction experts.

It is true only in a very broad sense that operating expenses vary as the train-mileage. This paper was prepared in moments snatched from very exacting work, but within a very few days after the paper

FIG. 1.—MASONRY, AND ROCK EXCAVATION, HICKORY CREEK CROSSING NO. 7. LOOKING SOUTH.

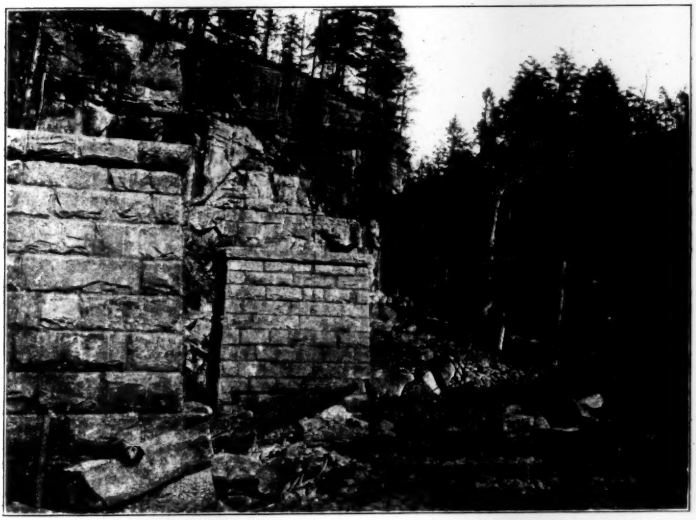


FIG. 2.—MASONRY, AND ROCK EXCAVATION, HICKORY CREEK CROSSING NO. 7. LOOKING NORTH.

(The line goes through the edge of the precipice.)

had been accepted for publication the writer noticed the error he had fallen into, and to which several of his critics have called attention. The calculation as to the value of the grade reduction was made anew, and, as the corrections were needed for the most part only in the figures, the writer held them, intending to make the necessary changes in the proof, which he, not being familiar with the mode of procedure in the Society's publications, expected would be sent to him. Mr. Taylor.

Now the saving that can be effected in operation by cutting down grades on a division which has been operating, say, 20 trains per day, so that the same business can be done by 16 trains, is a very different matter, for the first few years at least, from the saving that can be made in future operating expenses in constructing a road to do its business with 16 trains per day instead of 20. Every railroad manager knows how utterly impossible it is, when business falls off and the number of trains is reduced, to reduce expenses accordingly. The impossibility of accomplishing this is due primarily, but not entirely, to the fact that an organization equipped for the more expensive operation of 20 trains per day is now retained to operate a less number. The grade of service as to men, equipment, maintenance, etc., necessary to operate a division handling 20 trains a day is very much above the grade of the same required for 16 trains. When the number of trains is actually decreased in operation the more expensive organization is already in existence and will be dispensed with only with great difficulty and after long years, if indeed an increase of traffic does not warrant its retention. But in laying out a new road to handle its business with 16 instead of 20 trains per day the more expensive organization to operate 20 trains per day will not be developed.

This was the thought uppermost in the writer's mind. But its fallacy consists in the fact that a grade reduction is for all time, and the grade of the service, eventually, will be adjusted to the demands of the traffic.

Since this paper was prepared, this subject has been treated exhaustively in a paper* by Mr. J. B. Berry, Chief Engineer of the Union Pacific Railroad. Mr. Berry's analysis of the percentage of operating expenses on an operated road which can be saved by cutting down train-mileage through grade reduction shows a result of about 44. This is the lowest result that has been attained by any adequate analysis that the writer has seen, but, even if only this saving can be made by the grade reduction, it still was advisable, as Mr. W. P. Watson and Professor Raymond conclude.

The writer thinks there are good reasons for believing that Mr. Berry's and Mr. Watson's analysis of this percentage may not be quite conclusive in this case. For instance, in calculating the proportion

* *Bulletin No. 49*, American Railway Engineering and Maintenance of Way Association.

Mr. Taylor.

of operating expenses on which savings could be effected, both Mr. Berry and Mr. Watson assume that there would be no reduction in telegraph nor general expenses. Twenty trains per day will probably call for more train-order stations than sixteen, or will necessitate a night operator at stations where only day operators serve with the lesser number of trains. Wellington * says, of station, terminal, and general expenses: "There is nothing by which they are so quickly affected as by a decided increase in the number of trains."

The writer cannot see why the four fully-loaded trains on the 0.65% grade cannot just about handle the tonnage of six fully-loaded trains on the 1% grade. Supposing the trains to be pulled at 10 miles an hour by the identical engine described on page 5 of the paper entitled "Virtual Grades for Freight Trains," † the caboose to weigh 10.5 tons, the engine and caboose resistance to be 9 lbs. per ton, and the loaded-car resistance to be 5 lbs. per ton, the aggregate gross tonnage of loaded cars carried by the six trains on 1% grades will be 5 337, and that by the four trains on 0.65% grades will be 5 160. Mr. Beard seems to have overlooked the fact that when the grade is reduced the train load is increased by a larger percentage than is the percentage of grade reduction. As per the data above, the 35% reduction in this case allows an increase of 45% in train load.

The point made by Mr. Beard, however, in regard to the additional cost of reducing the ruling grade due to the slower speeds that heavier trains must make over the road, is well taken, but it seems a trifle ungenerous, under the circumstances, to hold that this will obtain for passenger trains and local freights.

One of the writer's critics contends that his annual cost of the pusher-engine service is too high, and another that it is too low. This is a matter of secondary importance, but the writer has the figures, made three years ago by the general superintendent of a western road, as to the actual cost of such a service where conditions were identical, in almost every respect, coal and water convenient, etc., with those contemplated for the pusher service on this road. The figures were furnished the engineering department of that road for use in calculating the value of a much-needed grade reduction. The annual cost given was $11 056. Further, Mr. Berry, in the Bulletin previously referred to, analyzes the cost per mile of pusher-engine service and finds it to be 34.4% of the cost of a train-mile for conditions obtaining on the Union Pacific Railroad. As fuel is very much cheaper near Jellico than on the Union Pacific, this is certainly a safer percentage on the Louisville and Nashville than in Wyoming. On this basis, the annual cost of this service on this line would be 34.4% of $1.08 × 32 544 engine-miles, or $12 090.

* "Economic Theory," Art. 716.

† *Transactions*, Am. Soc. C. E., Vol. L.

Mr Taylor.

Even when it would have helped him to make his point, in showing that the cost of the pusher service as fixed by the writer was too low, Mr. Beard does not seem to have calculated accurately the annual wages of a double crew, but adopted the writer's figures, which were for a single crew with reasonable allowance for overtime.

Really, Mr. Beard should revise his arithmetic. He should review his geography also, for he seems to suppose that the ore-carrying roads running into Duluth are in Michigan. And Duluth, the Duluth of Proctor Knott, is not a Michigan village.

The writer did not intend any invidious comparison between his own work and that of the unknown engineer who located the Knoxville and Ohio Branch of the Southern. The fact that the road is said to be one of the very best paying divisions of the Southern's system is sufficient to attest the worth of his work.

Neither did he intend to decry the work of the engineer who laid out the Chicago-Kansas City line referred to. The writer is familiar with every foot of the part of the line referred to, and, a few years ago, made some woeful failures in attempting to improve certain parts of this location.

In the comparisons made on grade smoothness, the rise and fall on minor gradients should very probably have been excluded. But since, on the three lines mentioned, so large a proportion of the rise and fall was made on the heavier and longer gradients that the result of the comparison would have been altered but little, the figures which were at hand were used. Each of the lines referred to has a heavier traffic than that immediately in prospect for the new line. The expensive construction necessary on this new line was warranted only because of the fact that the smoothness of the grades gave the owners a road which would admit of comparatively inexpensive operation.

Neither did the writer intend to intimate that any very wonderful engineering work had been done. The only wonderful thing about the matter was that a route, of the nature described, on which a road could be constructed at any outlay that the experienced executive officers of a first-class railway considered reasonable, existed at all in such a country. The writer is quite positive that the final result of the examination and surveys could not reasonably have been anticipated from any possible preliminary examination of the country and the available maps. Major R. H. Elliott, former Chief Engineer of the Kansas City, Memphis and Birmingham Railroad, acted as Consulting Engineer for the Louisville and Nashville, in the projection of this whole scheme. He had made a thorough study of the topographic maps, and was probably as familiar with the whole district as any man living. And yet, when the final result of the surveys was communicated to him, his reply was: "That 0.65% compensated grade is astonishing, and I did not know or think it possible."

Mr. Taylor. The conduct of the surveys and examinations was in no wise unusual. The topographic maps rendered unnecessary much of the examination which, otherwise, would have been essential. However, as the maps were somewhat inaccurate, and as the contours on these maps were at 100-ft. intervals, and the grade of the railroad so light, comparatively, a good deal of examination was necessary, particularly to determine the character of the country between the contours. The country was too heavily timbered, as a rule, to make much headway in the surveys by stadia methods, which, in suitable country, is a favorite method with the writer. Each party was equipped with aneroids, hand-levels, etc., and the party chief with a saddle-horse. Most of the reconnaissance work was done by the writer, or in conjunction with the chiefs of party. Sunday was a day very much in favor for a horseback ride across country. The writer, on more than one such day, took breakfast, dinner, and supper, each with different location parties, separated by a goodly number of miles. The company was as liberal as could be in regard to the number of men, engineering equipment, and the means for taking care of the men in camp. The writer had all the encouragement and co-operation from his superiors that he could possibly ask for, and was not hampered by any restrictions or requirements whatever. He was simply expected to secure the best line possible under reasonable outlay, and had authority to determine the location, fix the gradients and maximum curves, and to arrange for the letting of the construction contracts.

In conducting such a survey over an extended country, the writer has never believed in the system of having a central office control too minutely the surveys made by a number of parties; nor of hampering the party chiefs by picayunish requirements such as a report at the close of each day as to just how many stations of line have been run out. Such requirements reduce an engineer to a machine, and, by showing him that he is distrusted, deprive him of that self-confidence, and of the consciousness of the appreciation of his work on the part of his superiors, so essential to the best work. In this day of educated engineers, trained in loyalty as well as in efficiency, men can and always should be secured for such work who can be released from such petty exactions, and with whom it will pay to do it. The writer can recall many a day in survey camps of his own when the best progress was made and not a stake was driven. In such a survey party, it is well that the men who occupy the inferior positions should be engineers *in embryo*, men who have received at least a part of a college engineering course, so that, when rainy days come, or when the party is necessarily held up to work up notes, there will be no idlers in the camp.

Under the conviction that the best place for working out a survey in difficult country is in the camp where the men are doing the

work, each party was supplied with a draftsman and drafting equipment, and, instead of requiring a continuous stream of couriers bearing tedious maps, profiles and reports, passing back and forth between the camp and the central office, all such notes were kept in the hands of the chief of party until the survey was completed over the territory he was to cover, when a complete map, profile, estimate and report were made out and submitted by the men who had done the work. The writer believes that the requisite co-operation and unity of purpose between the parties was supplied by his continuous visits to the men in their camps, where he could talk over frankly and fully with each his special problems and difficulties, better than could possibly have been accomplished by any means whatever from a central office miles away from the work. Though the writer's office was in Knoxville, a large percentage of his time, during the location, was spent in the saddle. Mr. Taylor.

Of course, the number of parties that one man can look after in this manner is limited, but it gives the engineer in charge an insight into the character of the work as a whole, which he cannot possibly acquire behind a roll-top desk, or from the most elaborate maps and profiles spread out on a drafting table, miles and miles from the work and the men who are doing it.

It cannot be insisted upon too strenuously that the engineer in charge of such location should not be hampered at the outset (and, as already said, the writer was not) by the usual restrictions as to grade rates, curve radii, etc. And yet, in order that there may be unity in the results attained by several survey parties, tentative maxima should be fixed, by the engineer in charge, during the preliminary examinations and surveys; and, in order that there may be no careless work in using stiffer grades and curves than those demanded by the country, under the governing financial restrictions, the fact should be constantly kept before the party chiefs that these maxima are only tentative, and that subsequent developments may require the assumed maxima to be considerably reduced.

The line, originally, was reconnoitered between La Follette and Jellico with 1.25% grades in view, and a location through this district was made some seven years ago on this gradient. Surveys with the present construction in view were begun through this district on 1.15% grades in both directions. As already noted, the country was not only reconnoitered, but the original location surveys between Knoxville and La Follette were made on 1% grades in both directions; in each case with the final result which has been described. If Mr. Coverdale would intimate that this is a haphazard method which only just happened to secure good results in this case, the writer differs with him seriously as to the steps necessary to secure successful results in such work.

Mr. Taylor. The writer has never believed in the theory that difficult railway location must needs be done by a genius, or that there are men specially endowed with "an eye for country" to whom such work must always be entrusted. It is work in which training and experience go farther than inspiration, and in which nothing but hard work, common sense, and an intelligent comprehension of the ends in view will enable one to succeed. One cannot be sure that he has gotten the best line for his purposes between any two points in country at all difficult, until he has examined and studied thoroughly the entire area between, and until, in his mind's eye, its traffic possibilities are as clear as the possible revenue from his father's farm, and its outline as well defined as the contour of his mother's yard.

MEMOIRS OF DECEASED MEMBERS.

JAMES PETER BOGART,* M. Am. Soc. C. E.

DIED DECEMBER 24TH, 1903.

James Peter Bogart, son of John and Henrietta Candee Bogart, was born at New Haven, Connecticut, on February 28th, 1852.

His early education was obtained in the public schools of New Haven, and he completed his course in the Hillhouse High School in 1869. He then entered the Sheffield Scientific School of Yale University, taking the course in civil engineering, and receiving his degree in 1872.

After graduation he was employed by Mr. H. G. Scofield, of Bridgeport, Connecticut, in general engineering work for about three years. In 1875 he entered the service of the United States Coast and Geodetic Survey, and was employed for three years, under R. M. Bache, in charge of a plane-table survey of the region in the vicinity of New Haven. Upon the completion of this work, he returned to Bridgeport and again entered the employ of Mr. Scofield, where he remained for about three years, during which time a detailed survey and map of the entire city was made from which an official atlas of the city was printed.

In 1881, the Legislature of Connecticut appointed a commission to promote the new and rapidly increasing interests of the deep-water oyster-growing industry. This commission appointed Mr. Bogart to be its Engineer, a position which he filled for about eleven years. During this time he executed a very careful triangulation of the Connecticut shore of Long Island Sound from the Connecticut River to Greenwich, a length of about seventy miles; established an extensive system of signals and ranges for locating the boundaries of the oyster beds in the Sound, and surveyed and mapped 70 000 acres of oyster farms.

This was employment for which he was peculiarly well fitted. His natural thoroughness, together with the training he received in the service of the Coast Survey, made him exceedingly precise and painstaking in his work. Where such important and diverse interests as those relating to the establishment of boundaries between these oyster farms were concerned, Mr. Bogart's characteristic honesty, industry and carefulness were of special value.

During this period he also served as Engineer to the commission which, acting with a similar commission from the State of Rhode Island, established and defined the boundary line, below high-water mark, between these two States.

* Memoir prepared by Charles A. Ferry, M. Am. Soc. C. E.

In 1892 he bought out the business of Mr. Frank Bruen, who for several years had conducted an engineering office in New Haven, and took into partnership Mr. A. William Sperry, who had previously been Mr. Bruen's assistant. This partnership continued until 1899, when Mr. Sperry withdrew and Mr. Bogart subsequently conducted the office alone.

His familiarity with the boundaries of the oyster beds, the thoroughness with which he performed his work, and his strict integrity, created great demand for his services in cases of dispute, between owners of adjoining oyster beds, in settling the correct location of boundary lines.

His work lay principally along the line of surveying, rather than engineering, and was always executed with great care and with a high degree of precision.

Mr. Bogart was pre-eminently a man of domestic habits, and found his chief pleasure in his home and the society of his family.

Mr. Scofield, in whose employ Mr. Bogart spent so many years of his professional life, thus writes:

"He was a painstaking, conscientious workman, no detail being too small to receive careful investigation. Mr. Bogart, though naturally of a retiring disposition, was a courteous gentleman, a genial companion and a loyal friend; and, while fully recognizing the rights of others, was thoroughly insistent on the maintenance of his own."

He was a member of the Connecticut Society of Civil Engineers and also of the New Haven Chamber of Commerce.

In 1888 he married Miss Helen Day, daughter of Warren H. and Eliza H. Day, of Bridgeport, who, with two daughters, aged, respectively, fourteen and six years, survive him.

Mr. Bogart was elected a Junior of the American Society of Civil Engineers on January 4th, 1882; and became a Member on July 3d, 1895.

CHARLES EDWARD HENRY CAMPBELL,* M. Am. Soc. C. E.

DIED DECEMBER 6TH, 1902.

Charles Edward Henry Campbell, son of James and Mary Rose (Buxton) Campbell, was born at Summerside, Prince Edward Island, Canada, on January 7th, 1850. His father, who was of Scotch descent, was a native of Prince Edward Island, but his mother came from England, being a member of an old Derbyshire family.

Charles Campbell was educated at Prince of Wales College, Charlottetown, Prince Edward Island. At the age of twenty he went to Schuyler, Nebraska, where he learned the business of bridge building, first in the field as one of the erecting crew, and afterward in the office.

In 1874 he moved to Council Bluffs, Iowa, where he formed a partnership with Mr. E. W. Raymond, under the firm name of Raymond and Campbell, for the purpose of building bridges. The partnership continued until the end of 1886, and during its existence the firm built a great many highway bridges and viaducts throughout the western States, besides a number of structures for several railroads, among others the following:

Council Bluffs and St. Louis; Humeston and Shenandoah; Burlington and Missouri; Kansas City, St. Joseph and Council Bluffs; Minneapolis Union; Northern Pacific; St. Paul and Duluth; St. Louis, Des Moines and Northern; Cheyenne and Northern; Fremont, Elk Horn and Missouri Valley.

During 1887, 1888 and 1889 Mr. Campbell was western agent for the Milwaukee Bridge and Iron Works, when, among other structures, he built the Twenty-third Street Viaduct in Denver, Colorado; rebuilt the bridge over the Missouri River at Ft. Leavenworth, Kansas; erected a bridge over the Missouri River at Ft. Benton, Montana, and constructed several large bridges and viaducts in western cities.

In 1889 he formed a partnership with Mr. J. B. Marsh, of Des Moines, Iowa, under the name of Campbell and Marsh, as western agents for the King Bridge Company, of Cleveland, Ohio. This partnership lasted until 1892, and during its continuance the firm built a number of bridges for the Kansas City, Pittsburg and Gulf Railway, an elevated railroad in Sioux City, Iowa, and a Missouri River bridge at Cascade, Montana.

In 1889 he was retained as consulting engineer to design and superintend the construction of a bridge, about 1 000 ft. long, over the Loup River at Columbus, Nebraska.

In 1892 he took again the agency of the Milwaukee Bridge and Iron Works, retaining it until 1895, when his contract expired. During

* Memoir prepared by J. A. L. Waddell, M. Am. Soc. C. E.

this time, among other works, he built the temporary portion of the Omaha Bridge and Terminal Railway Company's bridge over the Missouri River between Council Bluffs, Iowa, and East Omaha, Nebraska.

From 1896 until his death he was engaged in contracting on his own account, constructing numerous small bridges in various places and a depot building for the Burlington road at Creston, Iowa.

In 1899 he was associated with Mr. James Saguin, in the construction of bridges for the Illinois Central Railroad Company near Council Bluffs, Iowa; and in 1901 he built a steel bridge over the Missouri River near Winston, Montana, and a combination bridge over the Snake River at Shelley, Idaho.

In 1885 Mr. Campbell married Miss Fannie Reeve, of Cleveland, Ohio, and she survives him. He left no children.

He was a prominent Mason and Knight Templar, and took great interest in everything relating to these societies, having held in them several high offices.

He was eminently of a genial and kindly nature, and won to himself many friends, who mourn his death sincerely. His business dealings were ever characterized by the utmost fairness; and he would have been a richer man, financially, had he made a practice of treating others as others treated him; but his disposition was rather easy going, so that he often failed to obtain his just dues, preferring to yield to others rather than to strive unpleasantly for his rights.

Had he been in any other line of the profession than highway-bridge contracting, he would have made his mark as a designer, for he often evolved excellent details to meet special conditions; but he had to make a living out of his business, and he could not do that and at the same time live up to his ideas in designing. He often complained bitterly of this. Occasionally, he had some small chance to show what he was capable of doing as a designer; but in most cases, even in his railroad-bridge work, there were restrictions of a financial character that prevented him from accomplishing anything out of the ordinary.

When given a contract, he generally managed to spend more money on the work in improving the character of the structure than he had originally estimated; consequently, he failed to become rich.

When allowed an opportunity to build first-class constructions, he always availed himself of it and succeeded in obtaining structures far above the common. For instance, the temporary piers, spans and trestles of the Omaha Bridge and Terminal Railway Company's bridge over the Missouri River at Council Bluffs are as fine a piece of timber construction as was ever built in America. The work has already outlived its allotted time by two years, and could be relied upon to last several years longer, were there any necessity therefor. It

is now being replaced with permanent construction, so that within a year this fine monument to Mr. Campbell's great constructive ability will no longer exist.

His death was due to a complication of diseases, resulting finally in Bright's disease. For more than a year he struggled bravely, almost without hope, and attended to his work long after he was too ill to go to his office or even to leave his chair.

He was a thoroughly good fellow, a true sportsman, an honest gentleman, and a broad-minded citizen, as well as a contracting engineer of high repute. His death at the early age of fifty-three is a loss to the engineering profession and to the citizens of the Middle West, among whom he was so extensively and favorably known.

Mr. Campbell was elected a Member of the American Society of Civil Engineers on October 3d, 1883.

BENJAMIN BURGH SMITH,* M. Am. Soc. C. E.

DIED FEBRUARY 8TH, 1904.

Benjamin Burgh Smith was the eldest son of Dr. Benjamin Burgh Smith and Catherine Smith Farr. He was born at Georgetown, South Carolina, on May 15th, 1835. His education was completed at the South Carolina Military Academy, from which school he was graduated in 1855.

Choosing Civil Engineering as a profession, he secured an appointment on the Engineer Corps engaged in the surveys of the Charleston and Savannah Railroad. His connection with this organization lasted upwards of two years, when young Smith decided to study medicine, influenced no doubt by the advice and example of his father. He attended the medical lectures in the South Carolina Medical College at Charleston, South Carolina, and was graduated in 1859 or 1860. During the intervals between the sessions of the medical College, he was employed on the construction of the Charleston and Savannah Railroad, in charge of track laying.

The opening scenes of the Civil War soon attracted Dr. Smith, and he promptly abandoned his profession and entered the service of the Confederate States. Into this service he threw himself with all the ardor of an earnest nature, and was distinguished for his gallantry, good judgment and military skill. He attained the rank of Colonel and was included in the terms of the surrender of General Jos. E. Johnston to General Sherman.

Subsequently to the Civil War he resumed the practice of his profession at Adams Run, South Carolina. Having been left without means, he was not able at that time to purchase a horse, but illustrated his high character by walking long distances to practice among the poor.

The poverty of the section in which he lived forced him, after a time, to abandon the practice of medicine and to resume the practice of engineering. After being engaged in surveys of railroad lines of less importance, he joined the Corps of Engineers making the surveys and location of the Port Royal Railroad, and continued in this service until the completion of the road to Augusta, Georgia.

He then entered the service of the United States, and soon thereafter, obtaining the position of Assistant Engineer in the United States Light-House Establishment, he continued in that position until his death on February 8th, 1904.

To the duties of this position, which was practically his life work, he devoted himself with all his natural zeal and energy. His good

* Prepared by the Rev. Ellison Capers, Col. C. S. Gadsden and James P. Allen, M. Am. Soc. C. E.

judgment and intelligence gave great weight to his opinions in all matters relating to the engineering work of the Sixth Light-House District.

Col. Smith was a man of sterling qualities and strong character. He never married, but his beautiful home life among his relatives, to whom he was devotedly attached, will be held by them in lasting remembrance.

Col. Smith was elected a Member of the American Society of Civil Engineers on November 7th, 1888.

SELWYN MELLON TAYLOR,* M. Am. Soc. C. E.

DIED JANUARY 25TH, 1904.

Selwyn Mellon Taylor was born in Allegheny, Pennsylvania, on November 5th, 1862, and perished on January 25th, 1904, in a brave attempt to save human life after one of the most disastrous explosions in the mining history of the United States. His parents were C. C. Taylor and Elizabeth (Gamble) Taylor. He was married in 1891 to Mrs. Mary Nolan, formerly Miss Mary Zinn. Mrs. Taylor and his two sisters are the only immediate relatives surviving him.

The explosion which cost him his life occurred at the Harwick Mine of the Allegheny Coal Company, near Cheswick, Pennsylvania. Although Mr. Taylor had severed his connection with this company, as an engineer, almost two years before, he, immediately upon receipt of the news, hastened to the scene of the accident with all the available men in his office. None of the State mine inspectors could be reached, and he at once assumed command of the disorganized forces. All means of ingress to the mine were destroyed, both cages having been blown out, the steel head frame wrecked above the landing floor and the man-way in the air-shaft blocked with timber torn from the sides of the shaft; all showing that an explosion of terrific violence, which must necessarily have traversed the entire mine, had taken place at both the inlet and outlet of the ventilating air current.

Mr. Taylor's firm conviction, shared alike by all mining men present, was that all life in the mine was extinct. The ventilating fan proper, being set back from the air-shaft, had escaped serious injury, and the air current entering the mine was only interrupted for a brief period. After having a sheave wheel attached to the uninjured members of the steel head-frame below the landing-floor and suspending a sinking bucket, Mr. Taylor decided to reverse the direction of the air current and attempt an entrance of the mine by way of the hoisting shaft. Others volunteered for this service, and repeated attempts were made to dissuade him from entering the mine on account of his asthmatic condition, but to no avail. With one companion he made the first descent, not intending to go beyond the shaft bottom.

There, one man was found alive and was sent up by Mr. Taylor with a request for a small number of volunteers. He sent up a second request for the mine plan, but before it reached him he had, with two volunteers, penetrated the mine about 700 ft. At this point they were all overcome by foul air and Mr. Taylor was unable to retreat. When found he was dead, and the presence of an indentation in the forehead gave assurance that his death was instantaneous. His companions were saved.

* Memoir prepared by W. E. Fohl.

The man whom he found alive proved to be the sole survivor of about one hundred and seventy-five in the mine at the time of the explosion; and that he did survive, the subsequent examinations of the ruins showed to be little short of miraculous. It is presumed that this survival led Mr. Taylor to think that the conclusions reached before entering the mine were at fault, and that still others might be alive.

The motives that actuate the simplest line of human conduct can but rarely be clearly and accurately defined. But quick, decisive action was characteristic of this man. His daring spirit was evidenced by every move of his professional and business career. His tender heart was manifested by a host of benefactions. And his many friends feel that these characteristics held in abeyance his judgment and led him to his untimely death.

Although Mr. Taylor's life was short, his mining career was comparatively long and eventful. He entered the office of R. L. McCully, Civil and Mining Engineer, on April 1st, 1880, was admitted by him as partner on August 15th, 1883, and assumed charge of the mining work of the firm, which he retained up to the time the partnership was dissolved, in October, 1890. This was the period of his work in the field, and it was during this time, largely, that he acquired the intimate knowledge of the bituminous coal fields of Pennsylvania and the methods of their exploitation that, later, placed him in the first rank of mining engineers of the United States. In 1886 he wrote a portion of the Annual Report of the Geological Survey of Pennsylvania, describing the mining methods in use in the bituminous coal region of Western Pennsylvania, and in 1889 was appointed by Governor Beaver, of Pennsylvania, one of the two mining engineers who are appointed every four years to serve on the Board of Examiners for Bituminous Mine Inspectors. He also assisted with valuable counsel in the framing of the Act of 1893, revising the mine laws pertaining to the bituminous coal region.

From the spring of 1891 until the time of his death he was actively engaged in construction and consultation practice. His examinations and reports cover practically all the known bituminous coal areas of the United States, and, at the time of his death, one of his assistants, acting under his instructions, was engaged in the examination of a coal field in Southeastern Borneo for the benefit of English capitalists.

The individual mines which he developed are too numerous for mention in this memoir. His two principal monuments, in this class of work, are the developments of the Eureka Fuel Company, in the "New Klondike" region in Fayette County, Pennsylvania, since absorbed by the United States Steel Company, and that of the Pittsburg Terminal Railroad and Coal Company, in the immediate vicinity of Pittsburg. The former consists of four mines, 1 200 beehive coke

ovens, a pumping station and about 12 miles of pipe line; the latter consists of seven mines opened simultaneously for the production of steam and domestic coal. Both these developments are, in the strictest sense of the word, representative of their class. They, in common with all his work, are chiefly marked by the simplicity of the means used to attain the desired end; the combination and adaptation of old and well tried methods and the careful laying out of work in advance based on accurate surveys. The commercial success of his work has been largely due to these factors.

At various times in Mr. Taylor's career, he received tempting offers to devote his entire time to separate organizations. These he refused, preferring the greater freedom of his general practice. In the year 1900 an opportunity was presented to acquire a very favorably located coal field, about 25 miles from Pittsburg. In this, Mr. Taylor interested some few personal friends, and with them organized the Midland Coal Company. Dock facilities were secured at Cleveland, and a rapid and systematic development of the coal field was carried forward, with the result that, at the close of the year 1902, three first-class mines and 8 miles of railroad were built and in active operation.

The situation of the property, in connection with its first-class development and the shipping facilities which had been acquired at Cleveland, gave it such value in the eyes of the Pittsburg Coal Company that the entire improvements were purchased by them at a large figure, and the entire acreage of coal was turned over to them under a lease the duration of which was forty years, the terms being so couched that it practically amounted to a sale of the entire holdings of this company to the Pittsburg Coal Company.

During the progress of these negotiations Mr. Taylor had become interested in the exploitation of a large body of Freeport coal within 12 miles of Pittsburg. The value of this property, up to this time, had been unrecognized; but a systematic exploration, by drilling and otherwise, justified Mr. Taylor in his judgment that this hitherto unknown coal field was one of the most valuable and easily accessible bodies of coal that has ever been known in the vicinity of Pittsburg. This tract was partially developed at the time of his death, and is still in the hands of his estate.

Owing to the reputation Mr. Taylor had acquired in the operating end of the coal business, by his development, successful operation and advantageous sale of the Midland Coal Company, and the brilliant opportunity which he had grasped in the above-mentioned field of Freeport coal, overtures were made to him by prominent financial interests which eventually led to the projection of the National Coal and Coke Company, intended to operate the above-described Indianola coal property, as well as some 1 700 acres of exceptionally fine coking coal on the Monongahela River, 8 miles above Brownsville. Mr.

Taylor was chosen president of this company, and his was the influence which was counted upon to interest Pittsburg capital, and also to carry its development and operation to a successful conclusion.

Mr. Taylor's career covered a period of rapid growth in the importance of the engineer's connection with bituminous coal mining, and was largely instrumental in fostering this growth. In its beginning, the engineer was a rarity in the Pittsburg region; there was simply the surveyor making maps of mine workings with the direction of which he had nothing to do. At its close, the engineer is paramount, and few moves are made without his counsel.

As is the case with all good engineers, Mr. Taylor's office was one of the schools of engineering that publish no catalogue and make no address to the general public. But a large number of young men have taken either partial or complete courses in it. Many of them are holding positions of responsibility elsewhere, and all of them hold in remembrance his valued teaching and kindly attitude toward the beginner. His own education was acquired in the ward and high schools of Pittsburg, entering the latter at a very early age. His father was a prominent member of the Allegheny County bar, and, being by nature a profound student, with mathematical attainments far above the average, rendered valuable assistance to the son in the preparatory stages of his engineering work.

Mr. Taylor was elected a Member of the American Society of Civil Engineers on October 7th, 1903. For many years he had been a prominent member of the Engineers' Society of Western Pennsylvania, and was a member of its Board of Directors at the time of his death.

PAUL DAVIS CUNNINGHAM,* Assoc. M. Am. Soc. C. E.

DIED JULY 13TH, 1901.

Paul Davis Cunningham was born in Monroe County, Georgia, on November 27th, 1869.

He was an undergraduate of Emory College, Oxford, Georgia. From July to September, 1887, he was a Rodman on surveys for the Atlanta and Florida Railroad, and then, until February, 1888, Assistant to the Resident Engineer on the construction of that road. From April to December, 1888, he was Rodman and Leveler on surveys for the Tennessee Midland Railway. From April to October, 1889, he was Leveler on surveys for the Cumberland Valley Extension of the Louisville and Nashville Railroad, and, then, until February, 1890, Assistant to the Resident Engineer on the Big Stone Gap residency of that road. In June, 1890, he became Resident Engineer on the construction of the Decatur, Chesapeake and New Orleans Railroad, where he remained until September of that year. From October, 1890, until March, 1891, he was Resident Engineer on the construction of the Clarksville Mineral Branch of the Louisville and Nashville Railroad. He was next engaged as Leveler on the survey of the Upper Tennessee River for the Engineer Department, U. S. A., which work occupied his time from May to October, 1891. From February, 1892, until February, 1894, he was Transitman in charge of one of the topographical parties of the International Boundary Commission between the United States and Mexico, and from August to October, 1894, he was U. S. Assistant Engineer in charge of remeasurements under that Commission. From October, 1894, to March, 1896, his position was that of U. S. Assistant Engineer on the International (water) Boundary Commission, United States and Mexico, and then he became Engineer Clerk in the Southwest and Northwest Divisions of the Engineer Department, United States Army, which position he held until June, 1898. From July to September, 1898, he was Assistant Engineer with the Chief Engineer on the staff of the Major General commanding the Army in Porto Rico. He then became Principal Assistant Engineer, Department of Havana, Cuba. In 1899 he was Acting Chief Engineer in the latter department, and in December, 1899, became Chief Engineer of the City of Havana.

On July 10th, 1900, Mr. Cunningham, then in Havana, received the following telegram from General Anson Mills from Washington:

"May I recommend you for Consulting Engineer, Boundary Commission, salary thirty-six hundred a year and expenses?"

On July 14th he received the following telegram:

"You have been appointed Consulting Engineer, Mexican Water Boundary Commission, subject to acceptance of your resignation as

*Memoir prepared by E. C. Lewis, M. Am. Soc. C. E.

Assistant Engineer and Superintendent, Havana Engineering Department, by Military Governor of Cuba. I start for El Paso to-day. Hope you will join me as soon as practical.

"ANSON MILLS, Commanding."

Mr. Cunningham joined his corps in August, 1900. He was Chief of Party for the International Boundary Commission, his work being to follow the course of the Rio Grande from El Paso to the Gulf, a distance of thirteen hundred miles. The voyage was deemed hazardous, and for half the distance the party would be cut off from all communication with the rest of human kind.

He was drowned in the rapids of the Rio Grande, below Eagle Pass, on Sunday, July 13th, 1901, and his body was not recovered until the next Tuesday. The remains were brought to Nashville, Tennessee, and then taken to Shelbyville, where, on July 19th, they were laid to rest by the side of his mother, in beautiful Willow Mount Cemetery.

"Death loves a shining mark." Never was this adage more fully exemplified. Paul D. Cunningham was a shining mark in life and for death. Born of most worthy parents, given all the advantages the American youth desires or requires, he reached manhood thoroughly equipped for the battle of life. Cultured, refined, considerate, of splendid manner and attractive person, with fine social, professional and official position and an unsullied escutcheon, what a shining mark he was—so shining that death struck him, a long way off, full fifty years before his time.

Paul Davis Cunningham was elected an Associate Member of the American Society of Civil Engineers on March 1st, 1899.

WILLIAM WARD REED,* F. Am. Soc. C. E.

DIED JANUARY 10TH, 1904.

William Ward Reed, son of William Wyndham and Elizabeth Ingram (Smith) Reed, was born in Ashtabula, Ohio, on April 1st, 1824. He was of New England descent, his great grandfather, Colonel Seth Reed, a physician at Uxbridge, Massachusetts, having commanded a regiment at the battle of Bunker Hill. Colonel Reed, his wife and two sons were among the pioneer settlers of Erie, removing thence from Ontario County, New York, in 1795.

William Ward Reed was educated at the academies of Ashtabula, Ohio, and Erie, Pennsylvania. Here he acquired a good, practical education, but added largely to this in later life by his careful reading of general history and many branches of science, and especially by his studies in the higher mathematics, thus laying a broad foundation for his future profession as a civil engineer.

From early life Mr. Reed was active and energetic in whatever he undertook, whether as student or man of business, and was especially thorough in the studies and practical matters pertaining to his chosen profession.

His first service as a civil engineer was in the location and construction, in 1849–50, of the Erie and North East Railroad, now a part of the Lake Shore and Michigan Southern Railroad, and while engaged in this work he was advanced to the position of Assistant Civil Engineer. In September, 1851, he went to Canada, and was engaged for four years on the Ontario, Simcoe and Huron Railroad, between Toronto and Collingwood, Ontario. During the following year he was engaged in the construction of the harbor at Collingwood, and was next in charge of the construction of the Niagara Road, from Clifton to Niagara-on-the-Lake. During the next two years he was Contractor's Engineer on the Sarnia Branch of the Great Western Railroad.

On returning to his home in Erie, in 1858, he built by contract an aqueduct over the Elk Creek, on the "Pennsylvania Erie Extension Canal," a work of more than ordinary engineering difficulty. After completing this work he was elected Superintendent of the same canal, in 1859, a position which he held until the canal was sold to the Pennsylvania Railroad Company, and by them closed and abandoned.

In 1867, when The Board of Commissioners for the construction of a system of water-works in the City of Erie was constituted, Mr. Reed was made a member of the Board, and subsequently was elected their President. He continued to hold this important position for twelve years, and it was under his administration that the works were built

* Memoir prepared by A. H. Caughin, Esq.

and the first pump installed; and, later, his engineering skill and experience were called into play in the construction of the great reservoir on Twenty-sixth Street. Subsequently, he was engaged for a number of years, either as contractor or engineer, on various railroads, until he retired from business. He was one of the founders and original stockholders of the Second National Bank of Erie, and was for many years one of the Directors and, for a part of this time, Vice-President.

Mr. Reed, notwithstanding his busy life as civil engineer and constructor, was always a public-spirited citizen, and liberal in his support of charitable and benevolent enterprises. He was one of the Board of Managers of the Hamot Hospital, and was long a prominent member of the Masonic Fraternity. He was a member of St. Paul's Episcopal Church, of which he had been a Vestryman for more than twenty years. Deeply interested in the political questions and movements of the day, he was an active member of the Republican party and a pronounced advocate of the protective policy of that party; and for three successive terms he received the support of a large majority of the Republicans of his county as their nominee for Congress, but suffered defeat in the nominating convention of the district.

Mr. Reed was of quiet and reticent habits, but always positive and outspoken in his views when circumstances demanded their expression. Of a kind and generous spirit, he invariably won the affection and esteem of the men employed under him. He was never married, but in his home life was a most tender and devoted son and brother, and toward his associates in social and business life a faithful friend.

Endowed with a very retentive memory, and giving himself up, in the quiet retirement of his later years, to a diligent course of historical and scientific reading, he became almost an oracle of information on many subjects. His business judgment was so sound that he was frequently consulted in important matters, and his advice was highly regarded by his business associates. Of pure and upright character; resolute of will, self-poised and self-reliant, bearing resolutely the misfortunes that came upon him, through the dishonesty of others, in the latter portion of his business career, and enduring uncomplainingly the severe sufferings of his last days, William Ward Reed passed away, deeply mourned by all the members of his family and by the inner circle of his many attached friends, bearing with him the undiminished respect and esteem of the community which, in many capacities, he had long and faithfully served.

Mr. Reed was elected a Fellow of the American Society of Civil Engineers on December 20th, 1872.

TRANSACTIONS

OF THE

American Society of Civil Engineers.

INDEX.

VOLUME LII.

JUNE, 1904.

SUBJECT INDEX, PAGE 562.

AUTHOR INDEX, PAGE 565.

Titles of papers are in quotation marks when given with the author's name.

VOLUME LII.

SUBJECT INDEX.

AGRICULTURE.

"South African Irrigation." Francis Robert Johnson. (With Discussion.) 1.

BREAKWATERS.

"The Breakwater at Buffalo, New York." Emile Low. 73. Discussion: Thomas D. Pitts, William T. Lyle, D. E. Hughes, G. H. Raymond and George E. Fell, 198.

BRIDGES.

"The Substructure of Marsh River Bridge." Herbert J. Wild. (With Discussion.) 451.

CONCRETE.

— used in Buffalo Breakwater. 94 *et seq.*

DOCKS.

"Dock Improvements at Liverpool." George Cecil Kenyon. 36. Discussion: Alfred Noble, L. J. Le Conte and Emile Low, 65.

DOMES.

"Theory of the Spherical Dome with a Homogeneous Surface, and of the Framed Dome; also Notes on the Construction of Masonry and Metal — ." E. Schmitt. 262. Discussion: Irving P. Church and H. H. Wadsworth, 309.

DREDGING.

"A Desirable Method of — Channels through River Bars." S. P. Maximoff. 215. Discussion: Rudolph Hering, William W. Harts, E. L. Corthell, A. W. Robinson, L. J. Le Conte, F. B. Maltby, William Gerig and F. H. Hilliard, 238.

EXCAVATION.

"Freezing as an Aid to — in Unstable Material." James H. Brace. (With Discussion.) 365.

FOUNDATIONS.

"The Substructure of Marsh River Bridge." Herbert J. Wild. 451. Discussion: George B. Francis, 465.

FREEZING PROCESS.

"Freezing as an Aid to Excavation in Unstable Material." James H. Brace. 365. Discussion: George E. Thomas, D. E. Moran, E. L. Abbott, T. Kennard Thomson, Werner Boecklin, H. J. Campbell, A. Gobert and Charles Sooysmith, 437.

GRADES.

"The Location of the Knoxville, La Follette and Jellico Railroad, of the Louisville and Nashville System." W. D. Taylor. (With Discussion.) 467.

HARBORS.

"The Breakwater at Buffalo, New York." Emile Low. (With Discussion.) 73.

IRRIGATION.

"South African —." Francis Robert Johnson. 1. Discussion: Augustine W. Wright and L. J. Le Conte, 27.

MEMOIRS OF DECEASED MEMBERS.

Bogart, James Peter. 545.
Campbell, Charles Edward Henry. 547.
Cunningham, Paul Davis. 556.
Reed, William Ward. 558.
Smith, Benjamin Burgh. 550.
Taylor, Selwyn Mellon. 552.

RAILROADS.

"Method Used by the Railroad Commission, of Texas, under the Stock and Bond Law, in Valuing Railroad Properties." R. A. Thompson. 328. Discussion: E. L. Corthell, F. Lavis, W. H. Coverdale and W. D. Taylor, 346.

"The Location of the Knoxville, La Follette and Jellico Railroad, of the Louisville and Nashville System." W. D. Taylor. 467. Discussion: Emile Low, William P. Watson, E. J. Beard, Walter Watson, William G. Raymond, F. Lavis, W. H. Coverdale, Harvey Linton and W. T. Forsythe, 490.

RAINFALL.

"South African Irrigation." Francis Robert Johnson. (With Discussion.) 1.

RIVERS.

"A Desirable Method of Dredging Channels through River Bars." S. P. Maximoff. (With Discussion.) 215.

SHAFT SINKING.

"Freezing as an Aid to Excavation in Unstable Material." James H. Brace. (With Discussion.) 365.

SURVEYING.

"The Location of the Knoxville, La Follette and Jellico Railroad, of the Louisville and Nashville System." W. D. Taylor. (With Discussion.) 467.

TRANSPORTATION.

— on Russian rivers. 215.

AUTHOR INDEX.

ABBOTT, E. L.
Freezing as an aid to excavation. 442.

BEARD, E. J.
Railroad location. 505.

BOECKLIN, WERNER.
Freezing as an aid to excavation. 443.

BOGART, JAMES PETER.
Memoir of. 545.

BRACE, JAMES H.
"Freezing as an Aid to Excavation in Unstable Material." 365.

CAMPBELL, CHARLES EDWARD HENRY.
Memoir of. 547.

CAMPBELL, H. J.
Freezing as an aid to excavation. 444.

CHURCH, IRVING P.
Spherical and framed domes. 309.

CORTHELL, E. L.
Dredging river channels. 240.
Valuation of railroad properties. 346.

COVERDALE, W. H.
Railroad location. 521.
Valuation of railroad properties. 349.

CUNNINGHAM, PAUL DAVIS.
Memoir of. 556.

FELL, GEORGE E.
Buffalo Breakwater. 205.

FORSYTHE, W. T.
Railroad location. 527.

FRANCIS, GEORGE B.
Substructure of Marsh River Bridge. 465.

GERIG, WILLIAM
Dredging river channels. 252.

GOBERT, A.
Freezing as an aid to excavation. 446.

HARTS, WILLIAM W.
Dredging river channels. 239.

HERING, RUDOLPH.
Dredging river channels. 238.

HILLIARD, F. H.
Dredging river channels. 254.

HUGHES, D. E.
Buffalo Breakwater. 202.

JOHNSON, FRANCIS ROBERT.
"South African Irrigation." 1.

KENYON, GEORGE CECIL.
"Dock Improvements at Liverpool." 36.

LAVIS, F.
Railroad location. 519.
Valuation of railroad properties. 348.

LE CONTE, L. J.
Dredging river channels. 249.
Liverpool dock improvements. 66.
South African irrigation. 30.

LINTON, HARVEY.
Railroad location. 526.

LOW, EMILE.
Liverpool dock improvements. 67.
Railroad location. 490.
"The Breakwater at Buffalo, New York." 73.

LYLE, WILLIAM T.
Buffalo Breakwater. 199.

MALTBY, F. B.
Dredging river channels. 250.

MAXIMOFF, S. P.
"A Desirable Method of Dredging Channels through River Bars." 215.

MORAN, D. E.
Freezing as an aid to excavation. 438.

NOBLE, ALFRED.
Liverpool dock improvements. 65.

PITTS, THOMAS D.
Buffalo Breakwater. 198.

RAYMOND, G. H.
Buffalo Breakwater. 203.

RAYMOND, WILLIAM G.
Railroad location. 517.

REED, WILLIAM WARD.
Memoir of. 558.

ROBINSON, A. W.
Dredging river channels. 247.

SCHMITT, E.
"Theory of the Spherical Dome with a Homogeneous Surface, and of the Framed Dome; also Notes on the Construction of Masonry and Metal Domes." 262.

SMITH, BENJAMIN BURGH.
Memoir of. 550.

SOOYSMITH, CHARLES.
Freezing as an aid to excavation. 447.

TAYLOR, SELWYN MELLON.
Memoir of. 552.

TAYLOR, W. D.
"The Location of the Knoxville, La Follette and Jellico Railroad, of the Louisville and Nashville System." 467.
Valuation of railroad properties. 353.

THOMAS, GEORGE E.
Freezing as an aid to excavation. 437.

THOMPSON, R. A.
"Method Used by the Railroad Commission, of Texas, under the Stock and Bond Law, in Valuing Railroad Properties." 328.

THOMSON, T. KENNARD.
Freezing as an aid to excavation. 442.

WADSWORTH, H. H.
Spherical and framed domes. 317.

WATSON, WALTER.
Railroad location. 510.

WATSON, WILLIAM P.
Railroad location. 497.

WILD, HERBERT J.
"The Substructure of Marsh River Bridge." 451.

WRIGHT, AUGUSTINE W.
South African irrigation. 27.